DuSable Park

Baratta, Bob to: Cathleen Martwick, Mary Fulghum 01/08/2008 08:07 PM

VERNETA SIMON, EUGENE JABLONOWSKI, kworthington,

Cc: terri.haymaker, cbhaller, kdickhut, nchueng, "Walsh III, Edward V.", "MacDonell, Don", "Kornder, Steve", "Vickery, Randall G."

Cathy and Mary,

Attached are the following two river wall reports:

1. River Wall Condition Assessment and Preliminary Repair Cost Estimate for Proposed DuSable Park Development prepared by Harza Engineering Company, Inc., dated March 2001; and (SDMS ID: 235981)

2. Underwater Investigation of the DuSable Park Dockwall prepared by Collins Engineers, Inc., dated April 2005. (SDMS ID: 348148)

As we discussed in our most recent meeting, the hoarding around DuSable Park (creating a buffer around the river wall of at least 17 feet) and precautions taken by the contractors ensure that no heavy equipment or material is placed within the area near the wall that may have a significant impact on the river wall. The contractors have taken into account the condition of the river wall in placing their equipment and other loads. For instance, the crane utilized by Case Foundation is set back 50 feet from the river wall, while the potential zone of influence around the river wall (based on conservative estimates) is no more than 15 to 20 feet.

While the reports indicate that the river wall is in need of repair and/or replacement, there is no indication that impacted soils have come into contact with the surface water. In fact, prior surface surveys, including the one conducted by STS in 2007 along the perimeter, do not indicate the presence of thorium-impacted material at or near the river wall. As you know, replacement of the river wall is a key component of the development of DuSable Park. We expect that work on the river wall will commence this year.

As always, please call me with any questions.

Bob Baratta

<<DuSableParkRiver Wall Condition Assessment.pdf>>
<<DuSableUnderwaterInvestigationDockwall.pdf>>

EPA Region 5 Records Ctr.

348150

RIVER WALL CONDITION ASSESSMENT AND PRELIMINARY REPAIR COST ESTIMATE FOR PROPOSED DUSABLE PARK DEVELOPMENT

Prepared for

Chicago Park District

By

HARZA
Engineering Company, Inc.

March 2001

EXECUTIVE SUMMARY

The Chicago Park District intends to initiate planning for the development of an unused parcel of land at the mouth of the Chicago River, currently known as "DuSable Park". The 3.5-acre site is bounded on the North, East and South sides by the Chicago River and Ogden Slip, with each of the sides comprising vertical sheet pile river walls. The West boundary of the property is formed by Lake Shore Drive. As a precursor to further planning initiatives the Park District requires information regarding the condition of the existing river walls and an estimate of the likely level of effort required to restore the walls to a condition suitable for the proposed use.

In 1997, Harza Engineering Company performed a condition assessment and structural analysis for the site under subcontract to Johnson, Johnson & Roy, Inc. The purpose of this present study is to prepare a supplemental report that describes the current conditions and to present conceptual level cost estimates for typical repairs that may be required before a park can be constructed. Comparative analyses have been prepared to estimate the potential impact of several development concepts on the existing river wall.

A condition assessment of the existing river wall was conducted as part of the current study. The inspection consisted of examining only the above water portion of the structure from the landside. No underwater inspection, or water-based inspection was performed. The condition assessment found that the southeast corner and a portion of the north wall were most in need of repairs. The needed repairs were divided into three main groups; partial wall replacement, wall stabilization and wall patching. The approximate extent and cost of each type of repair was assessed. In addition a cost estimate for entire replacement of the existing wall with a new wall was prepared. The following table summarizes the cost of the proposed repairs.

Location	Type of Repairs	Estimated Cost*
South Wall	Wall Patching	$ 18,000
Southeast Corner	Wall Replacement	$ 235,000
East Wall and Northeast Corner	Wall Patching	$ 21,000
North Wall	Wall Stabilization (30 ft.) and Wall Patching	$ 63,000
Common Items	Mobilization, Demobilization, Temporary Facilities and Demolition	$ 38,000
	Total (Minimum Recommended Repairs)	**$ 375,000**
	Total Wall Replacement (approx. 1,100 feet)	**$ 2,500,000**

Table ES - 1 Summary of Recommended Repairs and Cost Estimates

A park development concept (supplied by others) for the site was selected for analysis purposes. No inference should be made as to the desirability of this particular plan, or the schedule for development. The concept is for a park development that substantially re-grades the existing site. The purpose of these analyses was to define and quantify general development parameters that minimize negative impacts and/or maximize positive impacts to the river wall.

Due to the number of unknown parameters regarding the river wall (including depth of embedment, subsurface condition, and initial construction) development scenarios that do not increase the load on the existing wall are recommended. If development scenarios that increase the loads on the wall are preferred, significant modifications and/or wall replacement will be required depending on the proposed park configuration.

* These cost estimates are based on the assumption that no significant new loads are applied to the river walls. Section 7 contains further discussion regarding the cost implications of load variations.

RIVER WALL CONDITION ASSESSMENT AND PRELIMINARY REPAIR COST ESTIMATE FOR PROPOSED DUSABLE PARK DEVELOPMENT

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FOREWORD

Authorization

This study was completed under the authority of Designer/Consultant Agreement (Specification No. 98194) between the Chicago Park District and Harza Engineering Company, Inc. (Project No. 15442C).

Scope

The purpose of this report is to summarize the condition assessment of the DuSable Park river wall, evaluate the potential impacts to the river wall due to a proposed development scenario and to prepare cost estimates for repairs to the wall necessary to support the proposed development scenario.

Acknowledgments

This report was prepared on behalf of Robert Megquier, Director of Planning and Development and Mitch Glass, Project Manager, Chicago Park District. The report was prepared by or under the supervision of Mark Wagstaff, Project Manager, and Mark Calvino, Midwest Regional Manager of Infrastructure, Harza Engineering Company, Inc.

1.0 INTRODUCTION

The Chicago Park District (Park District) is considering park development alternatives for a 3.5-acre parcel of land bordered by the Chicago River on the east and south, Ogden Slip to the north and Lake Shore Drive on the west. The parcel is referred to as DuSable Park. The south, east and north perimeters comprise vertical steel sheet pile river walls. Further details of the site are included in Section 2, Site Description. As a precursor to the initiation of the development process the Park District requires information regarding the condition of the river walls and the potential costs associated with repairs that may be needed.

Harza Engineering Company, Inc. has performed a previous condition assessment of the river walls (DuSable Park Development, River Wall Condition Assessment, 1997), which included surface inspection, underwater inspection and engineering analysis. Further details of the 1997 work are included in Section 3, Summary of Previous Condition Assessment. The present report is intended as a supplement to the work performed in 1997. The present study was authorized to attempt to identify areas of the site that have deteriorated significantly since the last inspection and to examine the potential impacts of different development scenarios. To this end a site inspection was carried out. Further details of the site inspection are included in Section 4, Findings of the 2001 Condition Assessment.

As yet no plan for DuSable Park has been formalized and the schedule and nature of any development is not established. One of the determining factors will be the level of effort required to restore the site to a condition suitable for supporting a park development. This report presents a set of repairs and maintenance measures that are required to enhance safety before any public use of the parcel can be implemented. Details of the proposed repairs are presented in Section 5, Proposed Repairs. In addition to the recommended repairs, cost estimates for the repairs and maintenance measures have been prepared. Details are presented in Section 6, Cost Estimates.

An engineering evaluation of the existing wall, of potential stabilization measures and of a schematic development scenario was also performed. This analysis was performed by developing a set of baseline design parameters and comparing increases (or decreases) in the values for the different cases. Further details of the engineering evaluation are included in Section 7, Engineering Evaluation. Section 8, Summary, Recommendations and Conclusions completes the report.

Detailed breakdowns of the cost estimates are included in Appendix A, and results of a life-cycle cost analysis comparing partial or complete wall replacement is included as Appendix B. Details of the Engineering Evaluation is included as Appendix B.

2.0 SITE DESCRIPTION

The site covers an area of approximately 3.5 acres at the mouth of the Chicago River. The site is bounded on three sides by water, the Chicago River to the south and east, and Ogden Slip to the north. The western boundary of the study site is defined by Lake Shore Drive. Exhibits 1 and 2 show the project location and a site plan. The total length of the river wall around the site is approximately 1,100 lineal feet. Currently there are no above ground structures on the property. A more detailed description of the history and previous uses can be found in Harza's 1997 report, which drew heavily from a report titled "Environmental Reconnaissance" prepared by STS Consultants for the Chicago Dock and Canal Trust, 1989. At the time of the 1997 report efforts were made to obtain record drawings of the existing sheet pile river wall, but without success. Similar efforts in 2000/2001 have also been unsuccessful.

The geotechnical conditions, as described in the 1997 report (based on previous subsurface exploration at neighboring locations), indicate that the site comprises: dense silty clay overlain by medium stiff clay, overlain by soft clay, with silty sand fill materials comprising the upper layer. More detailed geotechnical information can be found in the 1997 report and the material referenced in that report.

The Metropolitan Water Reclamation District (MWRD) of Greater Chicago operates sluice gates located close to the project site that regulate the Chicago River water levels with respect to Lake Michigan water levels. MWRD keeps records of the river water levels at the Chicago Lock, which is to the east of the project site. Based on conversations with staff of MWRD, the Chicago River water level is generally maintained between –2.0 and –0.5 feet Chicago City Datum (CCD). Subsequent to extreme rainfall events the water level in the river rises. Over the past 32 years the water level has exceeded +3.0 ft. CCD on two occasions. The highest recorded water level is reported as +4.1 ft. CCD, which occurred during July 1996.

The South Wall of the project site constitutes the north bank of the Main Branch of the Chicago River. According to the National Oceanic and Atmospheric Administration's (NOAA) navigation map of the Main Branch of the Chicago River channel is maintained at a depth of 21 feet at mid-channel. The main branch in the vicinity of the project site forms a part of the Chicago Harbor, which falls under the jurisdiction of the U.S. Army

Corps of Engineers. The depth indicated on the navigation chart in the vicinity of the East Wall and the North Wall varies between approximately 15 and 19 feet. More detailed information including the water depth at the sheet pile, the depth of soft material at the sheet pile toe and the approximate slope of the river bed perpendicular to the sheet pile wall can be found in the 1997 report. No measurements of water depth were made as part of this present study.

The site plan (Exhibit 2) has been prepared based on a 1994 topographic survey performed by the Chicago Park District. No survey measurements were made as part of this study. However, the general topography of the site resembles that indicated on the 1994 survey. In general the ground surface slopes uniformly upwards away from each of the three river walls. The elevation of the top of each wall is approximately +5.0, and the high points on the site are at approximately +22.0.

3.0 SUMMARY OF PREVIOUS CONDITION ASSESSMENT

In 1997 the entire river wall was visually inspected both above and below the water surface to document the condition of the wall, and to identify defects or conditions, which could adversely affect the integrity of the wall. A systematic inspection program for collecting and documenting the inspection data was developed. This included division of the wall into quadrants using field established station points, the development of inspection procedures and documentation sheets for both the surface and underwater inspections, and a complete review of the safety procedures and practices to be employed during the assorted field inspections.

The inspection carried out in 1997 concluded that the condition of the river wall was good. The visible portions of the sheet piles and the protruding ends of the tie rods did not appear to be in need of major rehabilitation. Local damage was reported at several locations. The 1997 report surmised that major rehabilitation and/or replacement of the entire wall was unwarranted at that time.

The defects and deterioration observed in 1997 included denting, puncturing and other damage to the sheet piles presumably from vessel collisions, intermittent corrosion, pockets of material loss immediately behind the sheet piles, bulging of a portion of the north wall approximately 8" to 12" out of alignment at the top of the sheet piles. In addition the channel cap placed along the top of the sheet piles was misaligned or missing in several places. The river wall at the southeast corner of the site was observed to be lower than at the other portions of the wall, with some doubt as to the presence of tie rods and the possible use of a different sheet pile section.

4.0 FINDINGS OF 2001 CONDITION ASSESSMENT

4.1 General Conditions

The current inspection was carried out on January 29, 2001. Air temperature was above freezing but up to three inches of snow cover was present. The inspection revealed that conditions were substantially similar to those described in the 1997 report.

4.2 Specific Conditions

4.2.1 South Wall

The majority of the south wall appears to be in generally good condition with the exception of a couple of severely damaged individual piles at around the midpoint of the wall. Since this wall forms the north bank of the Chicago River navigation channel it is more exposed to the potential for damage due to vessel collision than other portions of the wall. It is hypothesized that the individual pile damage along this wall has been caused by such collisions. In addition, this wall is known to have been used in the past as a mooring location for barges. A typical repair detail for these conditions has been developed and is described in Section 5. A more detailed survey from the waterside is needed to locate and size the complete extent of such damage.

4.2.2 South East Corner

The south east corner of the site was the most difficult to inspect due to the presence of large timbers across the top of the wall. In addition, facing timbers on the riverside of the wall prevented the inspection of protruding tie rods and other features. These same conditions were reported in the 1997 inspection. It appears that the south and east portions of the wall comprising this corner may have undergone some lateral movement. It appears that a more substantial repair may be required in this vicinity. A suggested repair detail has been developed and is described in Section 5. It is hypothesized that a contributing factor to the deterioration of this portion of the wall is the non-uniform stress concentrations and 3-dimensional soil-structure interaction that takes place at a 90-degree corner.

4.2.3 East Wall

The majority of the east wall appears to be in good condition. The pockets of material loss reported in 1997 were still evident, but there did not appear to be any significant increase in the size or extent of the holes. It is postulated that these holes may have been developed by a felled tree. No major rehabilitation of the east wall is required.

4.2.4 North East Corner

The sheeting comprising the northeast corner appeared to be in good condition and well aligned. The channel cap, however, showed signs of distress including horizontal separation of distinct portions of the cap and damage to bolts connecting the cap to the sheet piles. It is not clear if the distortion to the channel cap is indicative of movement of the supporting sheet piles, or has been caused by another action, possibly the surrounding vegetation. (In other segments of the site the channel cap has been clearly displaced by the action of vegetation.) Since there is no apparent defect in the wall, no major rehabilitation is suggested for this portion of the wall, however continued monitoring and observation is recommended.

4.2.5 North Wall

There is significant deterioration on portions of the north wall, including a section of wall that is bulging and several areas of pronounced corrosion. The conditions, as visible from the above ground inspection, do not appear to have worsened significantly from the time of the previous inspection. A suggested repair detail has been developed and is described in Section 5.

4.3 Future Inspections

Conditions of the DuSable Park River Wall appear to be substantially similar to those reported in 1997. To provide a further level of detail, additional inspection of the existing tie rods could also be performed by excavating test pits. The nature of the anchor wall or deadman could be performed as well as evaluation of the tie rods for signs of deterioration. Non-destructive measurement techniques could also be employed to quantify the extent of corrosion at particular locations.

5.0 PROPOSED REPAIRS

5.1 Introduction

This section describes repairs that are recommended for implementation before any development of the property opens the site to public use. The recommended repairs have been developed under the assumption that the topography of the site remains essentially the same as the current conditions, and that no structures are built on the site. Section 7 of this report describes an engineering evaluation of the river wall under various development scenarios, some of which increase loads on the river wall. The addition of significant load to the wall will result in more extensive repairs being required.

As a result of the most recent inspection a list of proposed repairs has been developed. The descriptions that follow are general in nature. The repairs can be divided into three main groups; wall replacement, wall stabilization and patching. Wall replacement refers to the removal of an entire segment of the existing wall and the installation of a new wall segment. Wall stabilization refers to the installation of additional support structures to relieve a portion of the load from the existing wall and to bring the existing wall into better alignment. Patching refers to the installation of steel plates over existing piles that have been damaged or severely corroded. Wall patching is intended to substantially decelerate material loss from behind the wall and/or wall deterioration. A plan showing the approximate extent of each type of repairs is included as Exhibit 3. Exhibits 4, 5, and 6 show additional details of the proposed repairs. The extent of each type of repairs has been approximated based on the above ground visual inspection, and the dimensions and member sizes are approximate. In general typical member sizes from other similar projects have been selected. Approximate cost estimates for each of the repairs are presented in Section 6. In addition to the structural repairs described, a variety of maintenance items are recommended for the entire length of the river wall. A more complete description of each wall and the proposed repairs follows.

5.2 South Wall

Wall patching is recommended for certain individual or groups of sheet piles throughout the south wall. A typical detail is shown in Exhibit 6. The repair requires surface preparation of the sound steel surrounding the area to be patched and the welding of a bent plate over the damaged area. The most serious damage is located about 180 feet east of Sta. 0+00. Several piles have been dented and require patching. A more extensive water side inspection is required to more precisely quantify the total area of patching required. The initial estimate is that approximately 100 square feet of patching will be required which corresponds to about 5% of the above water area of sheet pile.

5.3 Southeast Corner

The southeast corner of the property is in need of repairs. This portion of the wall is the most difficult to inspect because of the presence of large timbers. It is suggested that wall replacement be implemented at this corner. The approximate extent of the existing wall that requires replacement is 40 feet north and 40 feet west of the corner as shown on Exhibit 3. The proposed repair comprises the installation of a new wall and the demolition and removal of the existing wall. A section and detail of the proposed repair is shown on Exhibit 4. The repair detail comprises a new steel sheet pile wall, supported with batter piles as shown. At each end of the repair vertical H-piles are recommended to help tie the repair into the existing wall. A cost effective solution is to build a diagonal wall across the inside of the corner. Using this arrangement (and assuming that 40' x 40' corner is to be repaired) the replacement wall will be approximately 60 feet in length and will replace 80 feet of existing wall. This will result in the loss of about 800 square feet of the property (0.02 acres, less than 1%). Reconstruction of the existing corner with a diagonal wall is likely to be a preferable long-term solution and will better match the northeast corner as well as being about 20% cheaper than replacing the existing corner in-kind.

5.4 East Wall

The east wall of the site is in generally good condition and repairs are expected to be limited to patching of relatively small holes. The pockets of material loss adjacent to the wall should be filled with a granular backfill material (preferably a relatively lightweight fill). The initial estimate is that approximately 10 square feet of patching will be required which corresponds to less than 1% of the above water area of sheet pile.

5.5 Northeast Corner

No specific repairs are proposed for the northeast corner. However, as described in the previous section there is some evidence of distress to the channel cap that could possibly have been caused by lateral movement of the wall. There is no obvious indication of major defects in the wall based on the visual inspection. Continued monitoring and observation is essential. If significant deterioration occurs a repair similar in nature and extent to those described for the southeast corner may be appropriate.

5.6 North Wall

A portion of the north wall is bulging northwards towards Ogden Slip. This condition was reported in the 1997 inspection and does not appear to be deteriorating rapidly. A repair detail is shown on Exhibit 5. The proposed repair entails driving new supporting piles landward of the bulging section that would act to relieve some of the load on the wall and could bring it back into alignment. Prior to installing the supporting piles the area immediately behind the wall would be excavated to an elevation below the tie rods, and broken or missing tie rods could be replaced. The extent of the repair is estimated to be about 30 feet. The existing condition of the tie rods (presently buried and not visible) may require a larger portion of the wall to be repaired.

Wall patching is recommended for certain individual or groups of sheet piles throughout the north wall, as previously described for the south wall. A typical detail is shown in Exhibit 6. The repair requires surface preparation of the sound steel surrounding the area to be patched and the welding of a bent plate over the damaged area. The most serious damage is located about 80 feet west of the northeast corner. A single pile has been cut and requires patching. A more extensive waterside inspection is required to more precisely quantify the total area of patching required. The initial estimate is that approximately 20 square feet of patching will be required which corresponds to about 1% of the above water area of sheet pile.

5.7 Recommended Maintenance Work

5.7.1 Fender Supports

The river wall has previously had a more extensive system of fenders attached to it. In many cases the fenders have been removed or deteriorated leaving the unsightly steel support frames and bolts. It is recommended that the remaining fenders, frames and bolts be removed and the resulting bolt holes in the sheet pile be plugged with an expansive epoxy grout. The purpose of the removal of the fenders is to discourage any mooring adjacent to the river wall, which may induce additional lateral loads on the wall. Removing the supporting steel will improve the appearance of the wall and plugging bolt holes will prevent material loss or vegetation growth through the holes.

5.7.2 Channel Cap

The channel cap that covers the top of the sheet pile is damaged, misaligned or missing in places. It is recommended that the remaining caps be removed and that a new, uniform channel cap be installed throughout the site. While serving no major structural purpose, the new channel cap will provide more uniformity to the river edge and a more pleasing river wall. In its current, deteriorated state the channel cap presents a tripping hazard to the public.

5.8 Typical Construction Sequence for Repairs and Maintenance Work

A suggested construction sequence for the repairs and maintenance of the river wall is presented below:

1. Survey of the existing wall. This task includes detailed alignment survey and waterside inspection.
2. Demolition of existing features such as damaged steel cap, bolts, protruding fender frames, fenders, miscellaneous steel, and mooring posts and concrete foundations.
3. Clearing and grubbing of a swath of land around the perimeter of the park (about 20 feet wide) and at the areas of major repair.
4. Excavation of fill materials behind the existing wall at the southeast corner and along the north wall.
5. Removal of existing steel sheet pile, walers, tie rods at the southeast corner to the dredge line. (This activity will be required to avoid creating any navigation hazards in the Chicago River channel.)
6. Drive batter piles and vertical piles at the southeast corner and along the portions of the north wall identified for repair.
7. Install new steel waler to driven piles.
8. Drive new steel sheet piles at the southeast corner.
9. Make all connections between the new support piles and the steel sheet piles.
10. Prepare surfaces of the existing steel sheet pile to receive the patching.

11. Install the new steel plate to patch the damaged and/or corroded sheet piles.
12. Plug bolt holes.
13. Install new channel cap.
14. Backfill excavated and existing holes behind the sheet pile, leaving the appropriate grade for the proposed landscaping treatment.

6.0 COST ESTIMATES

6.1 Introduction

The cost estimates presented in this section are intended to be at the conceptual level and should not be perceived as estimates of probable construction cost. Further engineering and geotechnical investigation is strongly recommended prior to finalizing the repair program. Two cost estimates are presented. The first covers the cost of implementing the recommended repairs as described in this report. A second estimate has been developed that encompasses a much larger scope of work including the entire replacement of the river wall.

6.2 Methodology

The cost estimates have been developed by estimating the various material quantities required to accomplish the repair tasks and concurrently estimating the corresponding unit price. Unit prices have been developed based on recent competitively bid projects of similar scope and through use of published data such as the Means estimating book. Estimates of items such as mobilization and temporary facilities have also been included. The estimated cost includes all of the work to repair and stabilize the river wall and to backfill excavated and existing holes. No attempt has been made to estimate the cost of further developing the site for safe public use.

6.3 Minimum Recommended Repairs

The estimated construction cost (at February 2001 price levels) of the recommended repairs is $ 375,000. This estimate includes 25% contingency which is appropriate for the current phase of design. No estimate for engineering, construction management or other owner's costs are included in this estimate. The material quantities and the assumed unit prices for this estimate are included in Appendix A. The estimate has been prepared based on the assumption that all of the proposed repairs are carried out at the same time. The cost may be greater if the repairs are performed in distinct phases. Similarly, the cost estimate assumes that construction will take place from the land-side. Water-based construction would likely be more expensive and could increase the overall construction cost by up to 50%.

The most significant cost items are the installation of new steel sheet piling and associated support members. If subsequent investigation (e.g. the preparation of a test pit) reveals additional defects not currently visible the cost of the proposed repairs will increase.

In order to illustrate the comparative cost of the individual repairs, the total cost of the repairs has been subdivided into individual estimates for each part of the site. The assumption, however, is that all of the work will carried out concurrently. Performing only a portion of the repairs may cause the costs to rise. The breakdown is tabulated below:

Location	Type of Repairs	Estimated Cost[1]
South Wall	Wall Patching	$ 18,000
Southeast Corner	Wall Replacement	$ 235,000
East Wall and Northeast Corner	Wall Patching	$ 21,000
North Wall	Wall Stabilization (30 ft.) and Wall Patching	$ 63,000
Common Items	Mobilization, Demobilization, Temporary Facilities and Demolition	$ 38,000
	Total	**$ 375,000**

Table 6-1 Breakdown of Minimum Recommended Repairs by Location

6.4 Complete Replacement of River Wall

The estimated construction cost (at February 2001 price levels) of replacement of the entire river wall is $ 2,500,000. This estimate includes 25% contingency, which is appropriate for the current phase of design. No estimates for engineering, construction management or other owner's costs are included in this estimate. The material quantities

[1] These cost estimates are based on the assumption that no significant new loads are applied to the river walls. Section 7 contains further discussion regarding the cost implications of load variations.

and the assumed unit prices for this estimate are included in Appendix A. The estimate has been prepared based on the assumption that the entire wall replacement is carried out at the same time. The cost may be greater if the replacement is performed in distinct phases. Similarly, the cost estimate assumes that construction will take place from the land-side. Water-based construction would likely be more expensive and could increase the overall construction cost by up to 50%.

This estimate has been prepared for comparative purposes only. Feasibility level investigations to determine an acceptable alignment for the new wall are required, as well as more detailed geotechnical characterization and surveying, and regulatory review and approval.

6.5 Life-Cycle Cost Analysis

One of the major differences between the two scenarios for which cost estimates have been prepared is the future requirement for maintenance and rehabilitation. All other factors remaining equal, a new wall can be expected to have a longer useful life than the existing wall and require less extensive maintenance. In order to assess the relative merits of implementing a repair program or a complete replacement of the wall, a life-cycle cost analysis can be used to evaluate the Net Present Value (NPV) of an investment based on an assumed discount rate and a series of future payments. For each scenario, a series of future repair and maintenance activities has been assumed and the dollar cost (2001 prices) of those activities.

6.5.1 Minimum Recommended Repairs

For purposes of comparison the following schedule of regular maintenance and repairs has been assumed based on carrying out the minimum recommended repairs described above.

- Annual land-based visual inspection;
- Underwater inspection and minor wall patching every 2 years;
- A similar set of repairs as those recommended in this report every 5 years;
- Approximately half of the entire wall repaired or rehabilitated after 10 years; and
- The entire wall is replaced after 20 years.

After the wall is replaced, the same schedule of maintenance and reapirs that is outlined below is assumed to be followed.

6.5.2 Complete Wall Replacement

For purposes of comparison the following schedule of regular maintenance and repairs has been assumed based on carrying out the complete wall replacement described above.

- Minimal visual inspection every 2 years;
- Underwater inspection every 5 years; and
- A similar set of repairs as the minimum recommended in this report after 30 years followed by the same inspection and maintenance program as described above.

6.5.3 Results

The preliminary life-cycle cost analysis was performed using discount rates of 6%, 8% and 12% and was extended over a time period of 50 years. The dollar values assumed for the different repair and maintenance scenarios are tabulated in Appendix A along with computed NPV values. The results of this preliminary analysis indicate that the minimum recommended repairs (followed by significantly more future maintenance expenditures) have lower NPV values than the entire wall replacement. As a result performing only the minimum repairs is a cost-effective course of action.

This analysis is intended to be illustrative in nature and does not account for the monetary value of the potential disruption to the park caused by more frequent repairs, nor does it assume any net increase in park revenues as a result of the completely replaced wall.

6.6 Evaluation of Advantages and Disadvantages of Partial or Complete Wall Rehabilitation

This report includes a series of measures, which could be implemented in order to allow the site to be developed as a public park, which would be cheaper than total wall replacement. Each alternative has several advantages and disadvantages. The following paragraphs are intended to present some of these potential advantages and disadvantages of each course of action.

6.6.1 Partial Wall Rehabilitation

Advantages:

- Less capital investment required before the determined use is implemented;
- Reduced construction time before the determined use is implemented; and
- Simpler regulatory approval process.

Disadvantages:

- Continued inspection and maintenance will be required;
- Proposed developments on the site are limited in the vicinity of the existing wall; and
- Future repairs may have to be performed from the water-side depending on the proposed site development.

6.6.2 Complete Wall Replacement

Advantages:

- Designed to current standards;
- Reduced regular inspection and maintenance schedules;
- Site-specific amenities (e.g. boat docking facilities) can be incorporated into the design;
- Less potential for disruption to the park during maintenance activities;
- Optimized to match park development; and
- More uniform appearance.

Disadvantages:

- Increased capital investment;
- Longer construction time before site can be developed; and
- More complex regulatory process.

7.0 ENGINEERING EVALUATION

7.1 Introduction

The 1997 report contains details of a structural analysis of the river wall that was performed to calculate the required depth of embedment needed for stability based on assumed parameter values describing the soil stratigraphy and properties, surface profiles and anchorage. Since the actual depth of embedment is unknown it is not possible to compute factors of safety for the river wall. The 1997 report concluded that while the wall "does not conform to current design philosophies and practices.....[it] is still standing and performing with the current loading conditions." The report recommended that loads greater than those present at the time not be introduced.

It is possible that future development plans for the site may include alteration and/or re-grading of the site to meet functional or aesthetic objectives. The potential impact of such changes is not known. The purpose of this section of the report is to develop quantitative guidelines to assist future planning efforts. For example, it is possible that it may become desirable to modify the existing topography. Since the lateral load on the retaining wall is a function of the height of the fill in the vicinity of the wall, such modifications may lead to instability of the wall.

7.2 Methodology

The selected methodology utilized for this evaluation comprised the following steps:

1. Establish baseline parameters and conditions to be used as a benchmark for evaluating proposed site alterations;
2. Analyze the baseline conditions using the US Army Corps of Engineers' (USACE) CWLSHT computer program. For given input data, the program will estimate 3 key design parameters: Depth of Embedment, Maximum Bending Moment and Anchor Force[2]. The results of the baseline case become the benchmark against which all subsequent results are compared.[3]

[2] Depth of Embedment is used to determine the tip elevation to which the sheet pile should be driven, Maximum Bending Moment is used to select the sheet pile section to be used for the wall, and the Anchor

3. Input data to describe alternative cases including the introduction of temporary construction loads, various re-grading alternatives and the influence of potentially stabilizing (or load reducing) modifications are collected and analyzed. For an individual parameter (e.g. magnitude of construction load), a range of values can be input to obtain an indication of the sensitivity of the analysis to that individual parameter.
4. The results of these cases (Depth of Embedment, Maximum Bending Moment and Anchor Force) are compared with the baseline case and used to determine the percentage increase (or decrease) in the parameters. This provides a quantitative scale to assess the potential impact of each alternative. In the tabulated results presented throughout this section negative values of variation indicate a condition that lessens the design parameters and equates to an increased factor of safety between the particular case and the baseline case. Positive values of variation indicate a condition that increases the design parameters and equates to a decreased factor of safety between the particular case and the baseline case.
5. Since the existing topography perpendicular to each of the walls and the potential modifications in the vicinity of each of the walls are different a baseline case has been developed for each wall (North Wall, East Wall and South Wall).

Descriptions of the cases and the results of the analyses are presented in the following sections. For each case the general trend of either increasing (or decreasing) the design parameters is the most significant result. The absolute magnitude of the increase (or decrease) is less reliable due to the uncertainty associated with the input parameters as discussed above.

Appendix A contains additional information about the model output including graphical depictions of the results.

Force is used to determine the size and spacing of the anchorage system, and the dimensions of other components such as the bolted connections.

[3] In this report, the absolute numerical value of the three computed parameters is disregarded since there is substantial uncertainty regarding the existing conditions. The City of Chicago, Department of Transportation and the US Army Corps of Engineers were contacted, but neither agency had any available record drawings.

7.3 Study Cases

The following sections describe the parameters and conditions assumed in each of the study cases. In general the cases that have been investigated can be divided into two broad categories; sensitivity analyses and proposed condition analyses.

The sensitivity analyses investigate the variation in wall design parameters based on the change of a single input parameter:

1. Replacement of existing fill with lightweight fill material;
2. Application of surcharge at the toe of the structure;
3. Lowering of the top of wall elevation; and
4. Application of temporary construction loads.

The proposed condition analyses investigate the variation in wall design parameters based on a hypothetical development scenario[4].

1. Re-grading of the site to facilitate a public park with connectivity to the mid-level of Lake Shore Drive.

7.3.1 Baseline Case

The baseline case assumes a horizontal ground surface at the top of the wall. The subsurface stratigraphy and properties are taken from the 1997 report without edit. A diagram illustrating this condition is shown in Exhibit 7. For each wall the elevation of the ground surface has been estimated from a 1994 topographical survey performed for the Park District. The following top of wall elevations were used in this study:

North Wall:	+5.7 feet
East Wall:	+4.9 feet
South Wall:	+5.0 feet

[4] A conceptual scheme developed by others has been used as prototypes. The use of this particular scheme in this analysis is for illustrative purposes only, and in no way should be interpreted as a recommendation from Harza or as indicative of the Chicago Park District's intentions for the site.

7.3.2 Existing Conditions

The existing conditions cases are the same as the baseline cases with the exception that they utilize representative sloping ground surfaces obtained from the 1994 topographical survey instead of the horizontal ground surface. Cross sections for each wall are shown in Exhibit 8.

Results obtained are tabulated below:

EXISTING CONDITIONS	% Variation from Baseline Case		
Parameter	North Wall	East Wall	South Wall
Embedment Depth	9 %	11 %	2 %
Bending Moment	5 %	7 %	1 %
Anchor Force	4 %	6 %	1 %

Table 7-1 Comparison Between Existing Conditions and Baseline Case

The positive values in Table 1 indicate that the actual ground surface (which slopes upwards away from the wall) has a minor adverse impact on the wall, as compared with the baseline case which assumes a horizontal ground surface.

7.3.3 Use of Lightweight Fill

For this case the landside fill is assumed to be replaced by a lightweight fill (e.g. blast furnace slag), which will reduce the lateral load placed on the wall. Two different values for the density of fill were investigated to illustrate sensitivity. This case was investigated to determine if the removal and replacement of the existing fill could be a feasible option for reducing the load on the wall in lieu of total replacement of the wall. Fill replacement was assumed to extend to elevation to the dredgeline (river bed).

Results obtained are tabulated below:

LIGHTWEIGHT FILL	% Variation from Baseline Case					
Parameter	North Wall		East Wall		South Wall	
	100 pcf	85 pcf	100 pcf	85 pcf	100 pcf	85 pcf
Embedment Depth	- 23 %	- 45 %	- 24 %	- 46 %	- 24 %	- 45 %
Bending Moment	- 16 %	- 33 %	- 17 %	- 33 %	- 17 %	- 33 %
Anchor Force	-15 %	- 30 %	- 15 %	- 31 %	- 15 %	- 31 %

Table 7-2. Comparison Between Lightweight Fill and Baseline Case

The negative values in Table 2 indicate that this alternative could potentially relieve the load on the wall. The extent of the fill replacement landward was not considered in this analysis, neither has the feasibility of performing this work been evaluated. The integrity of the tie rods and the anchorage would have to be maintained throughout the excavation and fill replacement. A conceptual-level cost estimate (at February 2001 price levels, with 25% contingency) for this work is approximately $ 1,000 per linear foot. A lesser volume of fill replacement would be cheaper, but would have a lesser load-reducing impact on the wall.

7.3.4 Addition of Surcharge at the Toe of the Structure

For this case stone surcharge is assumed to be placed on the riverside, which will serve to reduce the required depth of embedment of the sheet piles. Two different values for the surcharge (representing different volumes of stone placed at the toe) were investigated to illustrate sensitivity. This case was investigated to determine if the addition of surcharge could be a feasible option for stabilizing the wall in lieu of total replacement of the wall. Surcharge was assumed to be applied over a 15-foot wide strip adjacent to the wall. Extending the surcharge further into the river may interfere with navigation along the waterway. Discussions with the appropriate regulatory agencies would be required to determine if this is a feasible measure.

Results obtained are tabulated below:

TOE SURCHARGE	% Variation from Baseline Case					
Parameter	**North Wall**		**East Wall**		**South Wall**	
	2 ft.	5 ft.	2 ft.	5 ft.	2 ft.	5 ft.
Embedment Depth	- 14 %	- 31 %	- 14 %	- 32 %	-13 %	- 29 %
Bending Moment	0 %	0 %	0 %	0 %	0 %	0 %
Anchor Force	0 %	0 %	0 %	0 %	0 %	0 %

Table 7-3. Comparison Between Toe Surcharge and Baseline Case

The negative values for embedment depth in Table 3 indicate that this alternative could potentially reduce the required penetration of the sheet pile wall. The other design parameters are not impacted by the addition of toe surcharge. A conceptual-level cost estimate (at February 2001 price levels, with 25% contingency) for this work is approximately $ 400 per linear foot.

7.3.5 Lowering of the Top of Wall

For this case it is assumed that the top of the river wall be lowered by cutting the sheet piles and removing a layer of the existing backfill material. Since the anchorage system (tie rods) is located about 5 feet below the top of wall there is an opportunity to reduce the wall height by up to approximately 3 feet. The edge of the property would have to be designed to accommodate natural water level fluctuations, and possible overtopping by vessel wake. In this analysis the ground surface is assumed to be horizontal.

Results obtained are tabulated below:

WALL LOWERING	% Variation from Baseline Case					
Parameter	North Wall		East Wall		South Wall	
	EL. +4.0	EL. +2.0	EL. +4.0	EL. +2.0	EL. +4.0	EL. +2.0
Embedment Depth	- 13 %	- 31 %	- 6 %	- 25 %	- 9 %	- 27 %
Bending Moment	- 13 %	- 30 %	- 8 %	- 26 %	- 8 %	- 26 %
Anchor Force	- 23 %	- 46 %	- 13 %	- 39 %	- 15 %	- 40 %

Table 7-4. Comparison Between Lower Wall and Baseline Case

The negative values in Table 4 indicate that this alternative could potentially relieve the loads on the wall. A conceptual-level cost estimate (at February 2001 price levels, with 25% contingency) for this work is approximately $ 500 per linear foot.

Reducing the wall elevation may allow the development of a "softer" river edge treatment, with the addition of wetland type plantings. It would be technically feasible, but potentially expensive, to replace the existing anchors with a support system that would allow the top of the wall to be lowered to river level or below. The regulatory impact, and cost, of such a development is beyond the scope of this report, but would require investigation.

7.3.6 Temporary Construction Loads

For this case a uniform load is assumed to be placed on the landside, which will serve to increase the lateral load on the wall. Two different values for the load (representing different sizes of equipment) were investigated to illustrate sensitivity. In each case the load was assumed to be distributed over a 20-foot strip adjacent to the top of the wall. This case was investigated to determine if the staging of construction equipment close to the wall could have a detrimental impact on the wall.

Results obtained are tabulated below:

CONSTRUCTION LOADS	% Variation from Baseline Case					
Parameter	**North Wall**		**East Wall**		**South Wall**	
	250 psf	500 psf	250 psf	500 psf	250 psf	500 psf
Embedment Depth	28 %	190 %	24 %	81 %	21 %	64 %
Bending Moment	33 %	139 %	23 %	78 %	27 %	82 %
Anchor Force	30 %	61 %	32 %	64 %	32 %	64 %

Table 7-5. Comparison Between Construction Loads and Baseline Cases

The positive values in Table 5 illustrate the potentially dangerous impacts to the wall if heavy construction equipment is operated close to the wall. Specific weight restrictions are nearly impossible to determine due to the different load distribution characteristics of different equipment and the time-dependency of the soil response. In addition, since the parameter values are non-linearly related to the construction loadings, case specific analyses should be performed before allowing construction equipment onto the site and the responsibility of any construction contractor to maintain the integrity of the wall must be spelled out in any contract specifications. The impacts of the construction loads can be mitigated by the use of mats to spread loads over larger areas and by keeping large loads further away from the wall. Depending on the nature of the work being performed, temporary bracing of portions of the wall may also be considered.

7.3.7 Park Development

To investigate the potential impacts of a proposed park development (Exhibit 9, 10) each wall was analyzed based on the following presumed modifications:

1. Re-grading of the site as shown on the possible park development provided to Harza.

2. Introduction of loads associated with public art as shown on the possible park development provided to Harza.

Results obtained are tabulated below:

PROPOSED PARK	**% Variation from Baseline Case**		
Parameter	**North Wall**	**East Wall**	**South Wall**
Embedment Depth	229 %	0 %	10 %
Bending Moment	854 %	0 %	3 %
Anchor Force	11 %	0 %	1 %

Table 7-6. Comparison Between Proposed Park and Baseline Conditions

The results obtained for this case indicate that the proposed park development appears to have no measurable impact on the east wall and a only a minor impact to the south wall. However, the impact to the north wall is clearly significant, and changes to the proposal and/or additional stabilization measures should be investigated. The relationships among wall fill height, fill location (distance from the river wall) and impacts to the wall are non-linear. Moving the proposed retaining wall landward from the river wall, and/or reducing the height of the retaining wall would reduce the potential impacts. Concurrent design of the park development and river wall improvements provide an opportunity for optimization of both structures.

In Section 6 the cost of replacement of the entire wall was estimated to be on the order of \$2.5 million, which represents about \$2,300 per linear foot of wall. Based on the results obtained above, implementation of the proposed park development would require the North Wall to be rebuilt with a significantly larger steel sheet pile section embedded further into the ground with anchors spaced at more frequent intervals. A preliminary estimate of the additional cost of such a wall section is between \$2,000 and \$2,800 per linear foot. Assuming that the larger wall section would be required for a length of between 200 feet and 350 feet results in additional costs of between \$400,000 and

$1,000,000 to replace the North Wall. This would drive the cost of entire wall replacement to between $2.9 million and $3.5 million. The cost of repairing the East and South walls to the extent described in Section 6.3 and upgrading the North Wall to support the re-grading is estimated to be between $1.2 million and $2.1 million

If the proposed development is implemented as shown, the additional cost required to replace the North Wall could be mitigated by designing the new retaining wall structure to have its own independent foundation. Alternatively mechanically stabilized soil techniques could be used to reduce the lateral loads applied to the river wall. In addition to the potential impacts to the river wall, addition of significant surcharge to an area of the park may cause settlement of the underlying soils. While design of the park features is outside the scope of this supplemental report it is noted that considerable efficiencies could be obtained by concurrent detailed design of the park and river wall improvements.

8.0 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

A summary of the inspections, analyses and findings of this study are as follows:

1. A review of previous studies performed pertaining to the proposed DuSable Park site was conducted. The primary resource was a 1997 report prepared by Harza documenting a surface and underwater inspection and condition assessment;

2. A condition assessment of the existing river wall structures was conducted. The inspection consisted of examining only the parts of structures above the waterline from the landside. No underwater inspection, or water-based inspection of the site was performed;

3. A primary finding of the 1997 report was that the river wall in its existing condition would likely not meet current design guidelines. However, since the wall has had a satisfactory service life there is no reason to assume that the wall will not continue to perform adequately as long as no additional loads are introduced;

4. The condition assessment performed as part of this study did not reveal any evidence that the previous conclusion does not still apply;

5. The condition assessment found that the southeast corner of the site and a portion of the north wall are in need of the most extensive repairs. The majority of the wall is in relatively good condition;

6. A suite of suggested repairs has been developed. The repairs consist of three distinct types: patching of damaged and/or corroded sheet piles, installation of support piles to realign existing piles and total replacement of a portion of the wall;

7. Conceptual level details of each type of repair have been prepared, and an estimate made of the extent of each repair type;

8. A cost estimate for the construction of the river wall repairs have been developed, along with an estimate for the complete replacement of the river wall. The conceptual-level cost estimates are in February 2001 dollars and are subject to cost escalation and inflation.

9. The estimated cost of the minimum recommended repairs to the river wall are $375,000. The estimated cost of replacement of the entire wall is $2,500,000;

10. A life-cycle cost analysis indicates that the Net Present Value (NPV) of the proposed repairs is less than the NPV of the entire wall replacement (see Appendix A);

11. An engineering evaluation was performed of a proposed development scenario for the site. For comparison purposes key parameters regarding the design of the river wall were computed under existing conditions and under the proposed development scenario;

12. The proposed park development supplied by the Park District includes a new retaining located close to the existing north river wall. As shown in the conceptual design, this retaining wall will require substantial modifications to the existing river wall. A new wall in this location, designed to meet the requirements of the proposed grading may be twice as expensive (per linear foot) as a replacement wall designed to meet the requirements of the existing site grading; and

13. A preliminary estimate of the cost of replacing the entire wall to support the development as shown in the conceptual design is between $2.9 million and $3.5 million. Implementing the minimum recommended repairs for the East and South walls and upgrading the North wall to support the re-grading is estimated to be between $1.2 million and $2.1 million.

8.2 Conclusions

Conclusions based on the data analysis and evaluations presented in this report are as follows:

1. The DuSable Park river wall appears to be in substantially the same condition as was reported in 1997, the occasion of the most recent documented inspection;

2. The conclusion reached in the 1997 report that "minor rehabilitation is required if the operational/functional requirements and loads placed on the wall remain unchanged" is still justifiable at the present time;

3. If the existing wall is not replaced, development scenarios for DuSable Park that do not increase the lateral load on the existing river wall are recommended. However, development scenarios that increase the loading on the wall should not be automatically dismissed, but should include mitigative measures or wall strengthening measures;

4. While the three components of the river wall (South Wall, East Wall, and North Wall) are in relatively good condition, repairs ranging from wall patching to wall replacement for individual segments of the site are recommended prior to opening the site to public use;

5. Three stabilizing measures were investigated during the engineering evaluation. The percentage reduction in design parameters resulting from the stabilizing measures varied from 0% to 45%. Further investigation is required to verify whether any one of these options can be used individually or in combination to substantially offset modifications to the site;

6. Proposed development of the property which leaves the existing topography relatively unchanged within 30 feet of the river wall is unlikely to lead to major reconstruction of the wall being required;

7. Proposed development of the property which includes the construction of large fill-retaining structures close to the river wall will require major reconstruction of the river wall or the implementation of independent foundation treatments;

8. Future development will impact the ease with which future repairs can be carried out. Under the development scenario provided, access to the site may be restricted in the future, which may lead to water-side construction, which is generally more expensive.

8.3 Recommendations

Specific recommendations for the development of the site are listed below:

1. More detailed repair-specific geotechnical information is required before designs for the rehabilitation measures (and the associated costs) can be finalized;

2. If the existing wall is not replaced, or portions thereof, modifications to the site which involve elevating the existing ground surface in the vicinity of the river walls are not recommended. The appropriate setback distance is related to the proposed height change and should be evaluated on a case-by-case basis;

3. Construction of any development at the park should limit construction loads within 20 feet of the existing river walls;

4. No new lateral loads should be introduced to the river wall. Examples include the introduction of boat mooring locations and increasing the site elevation. If loading is increased then designs to account for this new loading should be developed;

5. Design of the site development proposals should be coordinated with the design of the river wall improvements to allow for the greatest level of optimization between the projects;

6. Regular inspection and maintenance of the river wall will be required under any development scenario. The frequency and level of detail of such inspections has will be dictated by the extent of the rehabilitation measures and the proposed uses.

REFERENCES

Harza Engineering Company, 1997, "DuSable Park Development; River Wall Condition Assessment Report", prepared for Johnson, Johnson & Roy, Inc.

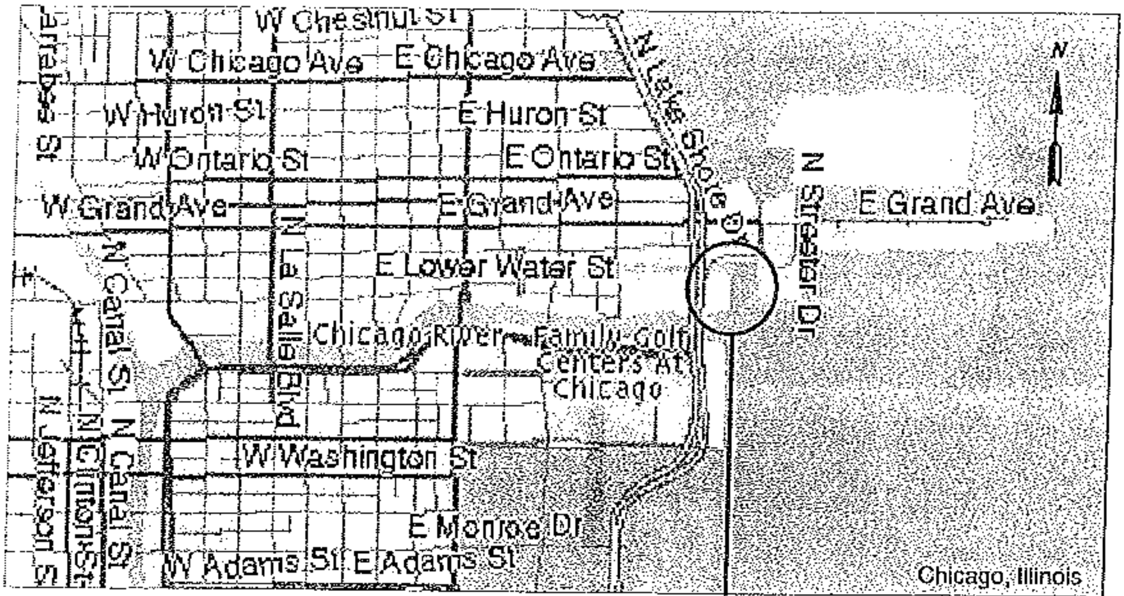
STS Consultants Ltd., 1989, "Environmental Reconnaissance, E. North Water Street and Lake Shore Drive Chicago, Illinois" prepared for Chicago Dock and Canal Trust.

Chicago Park District, 1994, "DuSable Park" Topographic Survey.

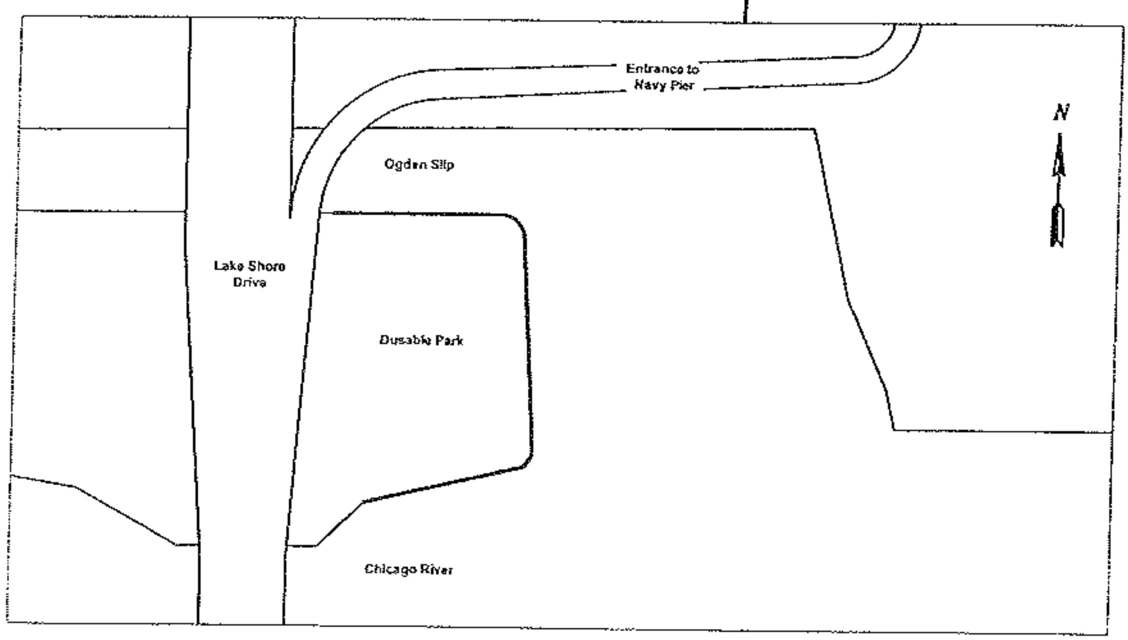
U.S. Department of Commerce, 1996, National Oceanic and Atmospheric Administration Recreational Chart 14926.

EXHIBITS

EXHIBIT 1
Project Location



VICINITY MAP

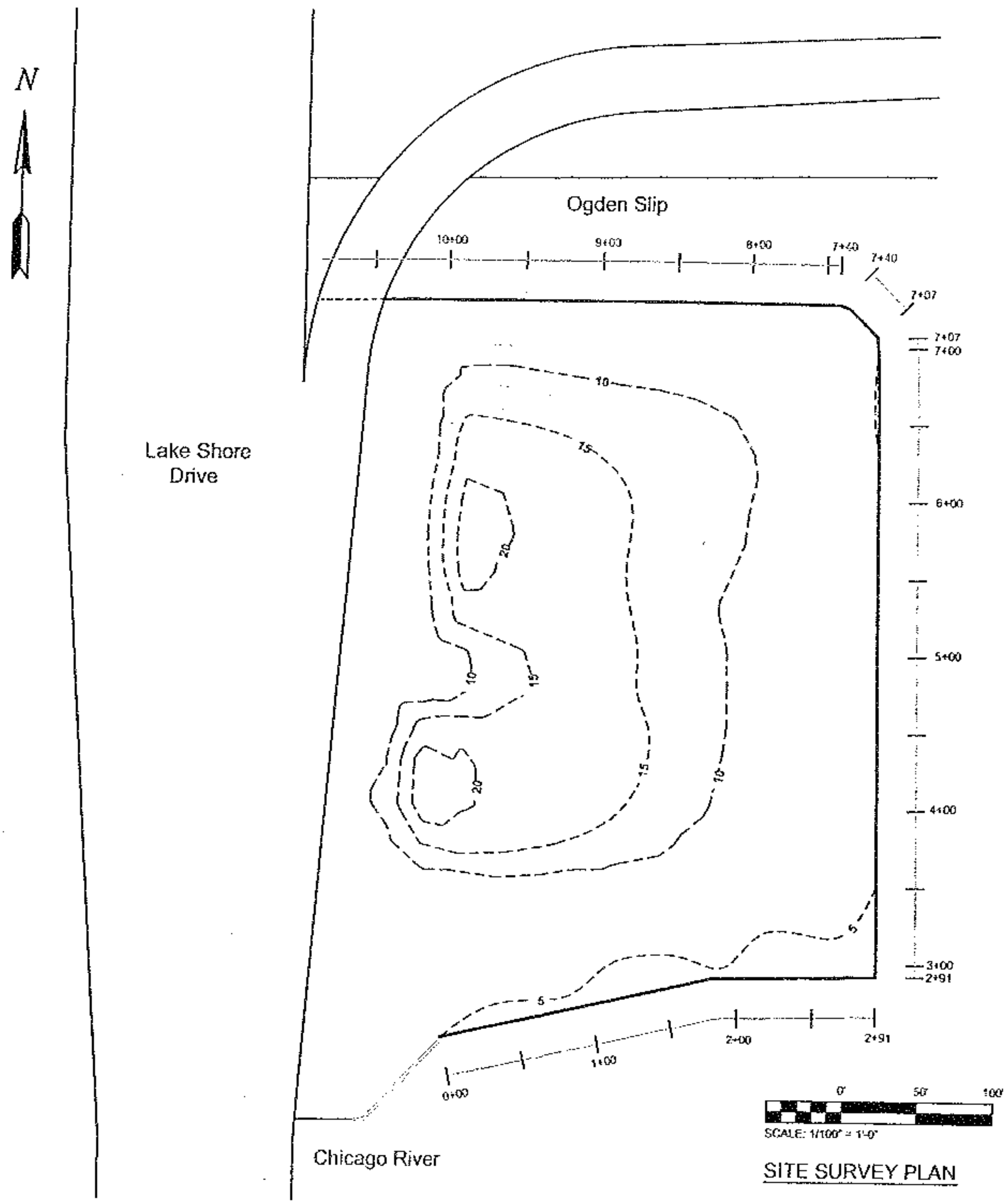


LOCATION MAP

DUSABLE PARK DEVELOPMENT
RIVER WALL CONDITION ASSESSMENT
FEBRUARY 2001

HARZA

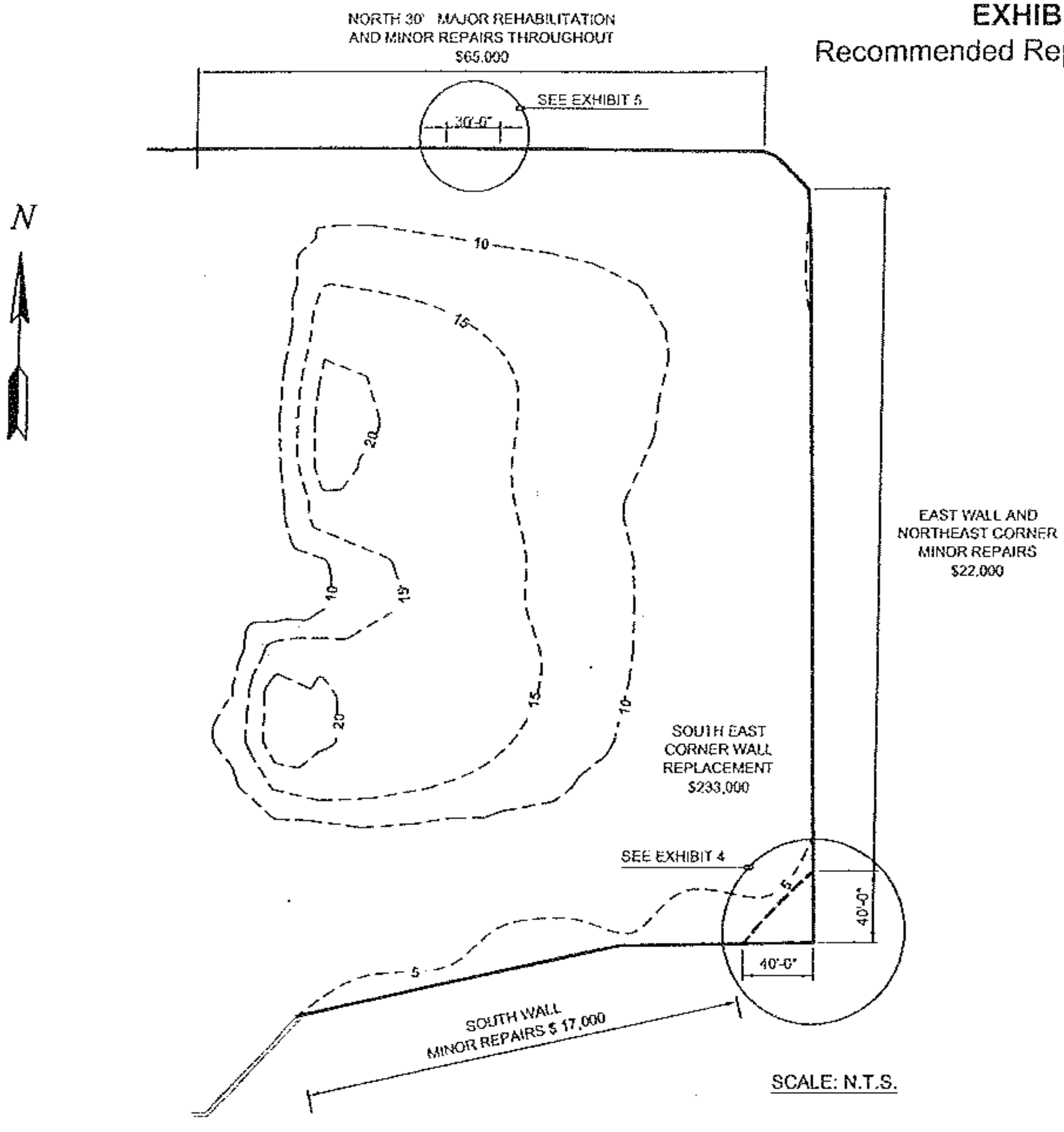
EXHIBIT 2
Site Plan
1994 Chicago Park District Survey



HARZA

DUSABLE PARK DEVELOPMENT
RIVER WALL CONDITION ASSESSMENT
FEBRUARY 2001

EXHIBIT 3
Recommended Repairs



SCALE: N.T.S.

NOTES:

I. All cost estimates are at February 2001 price levels and include 25% contingency.

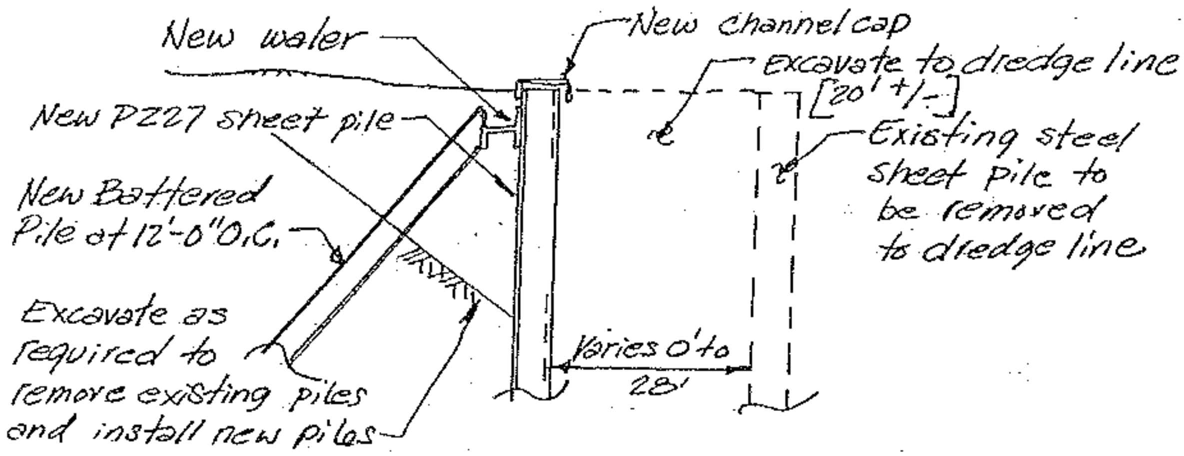
II. Mobilization, Demobilization, Temporary Facilities and Minor Demolition estimated to be $38,000.

III. Cost of repairs shown in this exhibit assume that no significant new load is added to the wall. Total cost of repairs shown is $375,000.

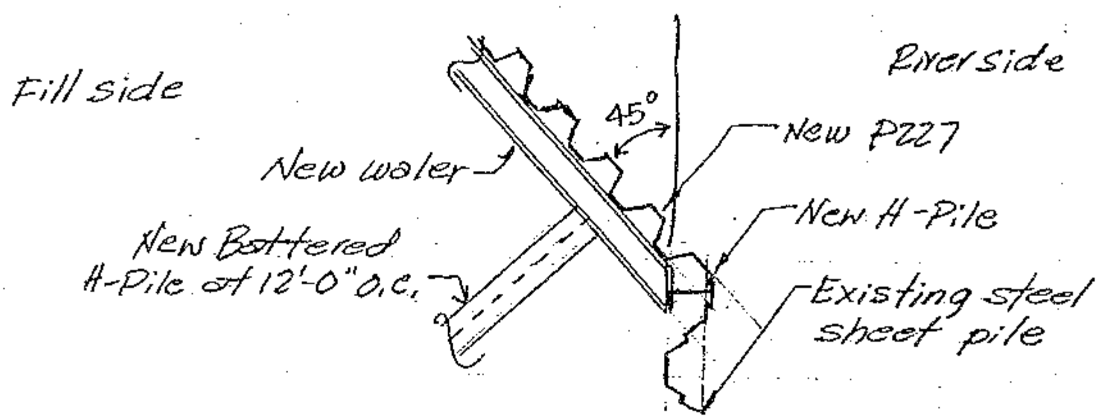
DUSABLE PARK DEVELOPMENT
RIVER WALL CONDITION ASSESSMENT
FEBRUARY 2001

HARZA

EXHIBIT 4
Southeast Corner Wall Replacement
Section and Detail

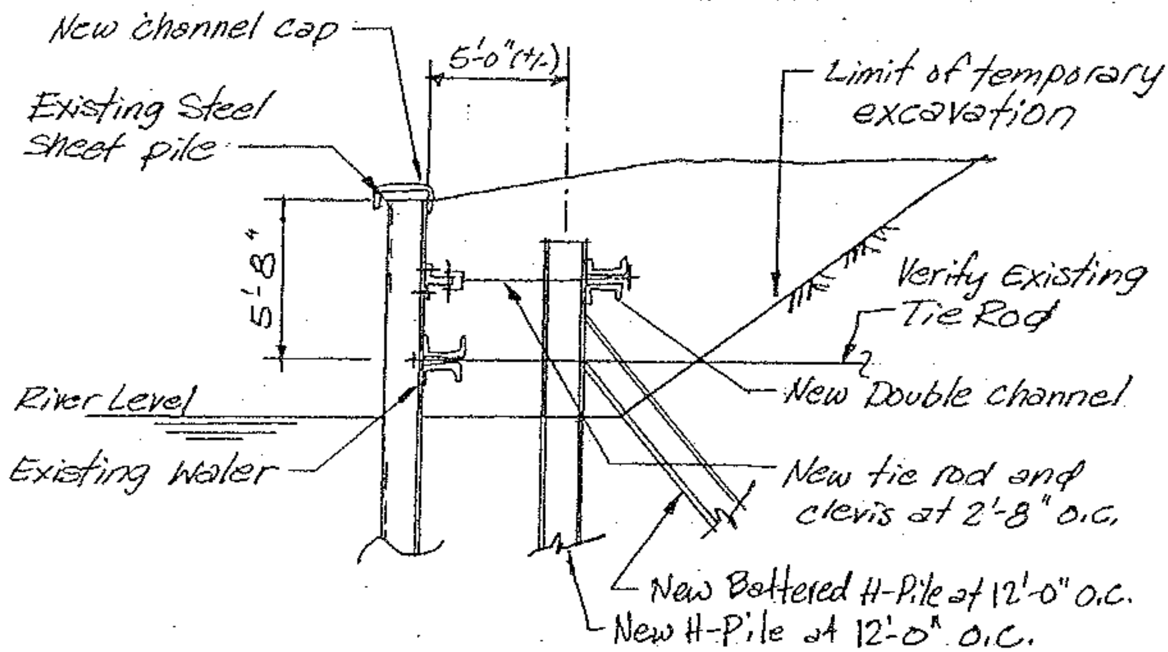


Proposed Repair for Southeast Corner
Typical Section



Proposed Repair for Southeast Corner
Typical End Detail

EXHIBIT 5
Proposed North Wall Stabilization Measures
Typical Section

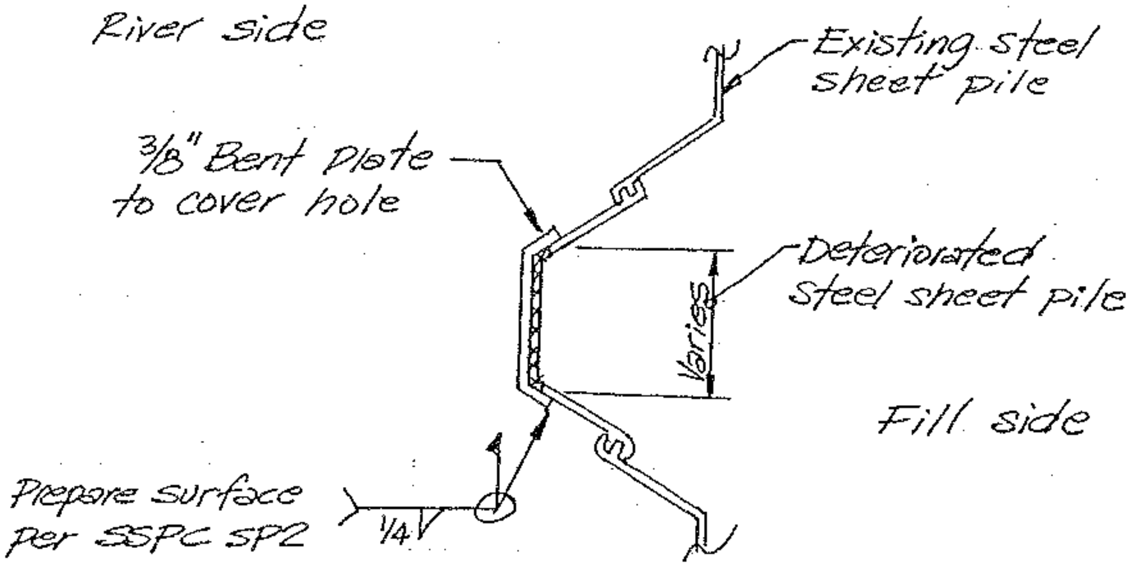


Proposed Repair for North Wall
Typical Section

DUSABLE PARK DEVELOPMENT
RIVER WALL CONDITION ASSESSMENT
FEBRUARY 2001

HARZA

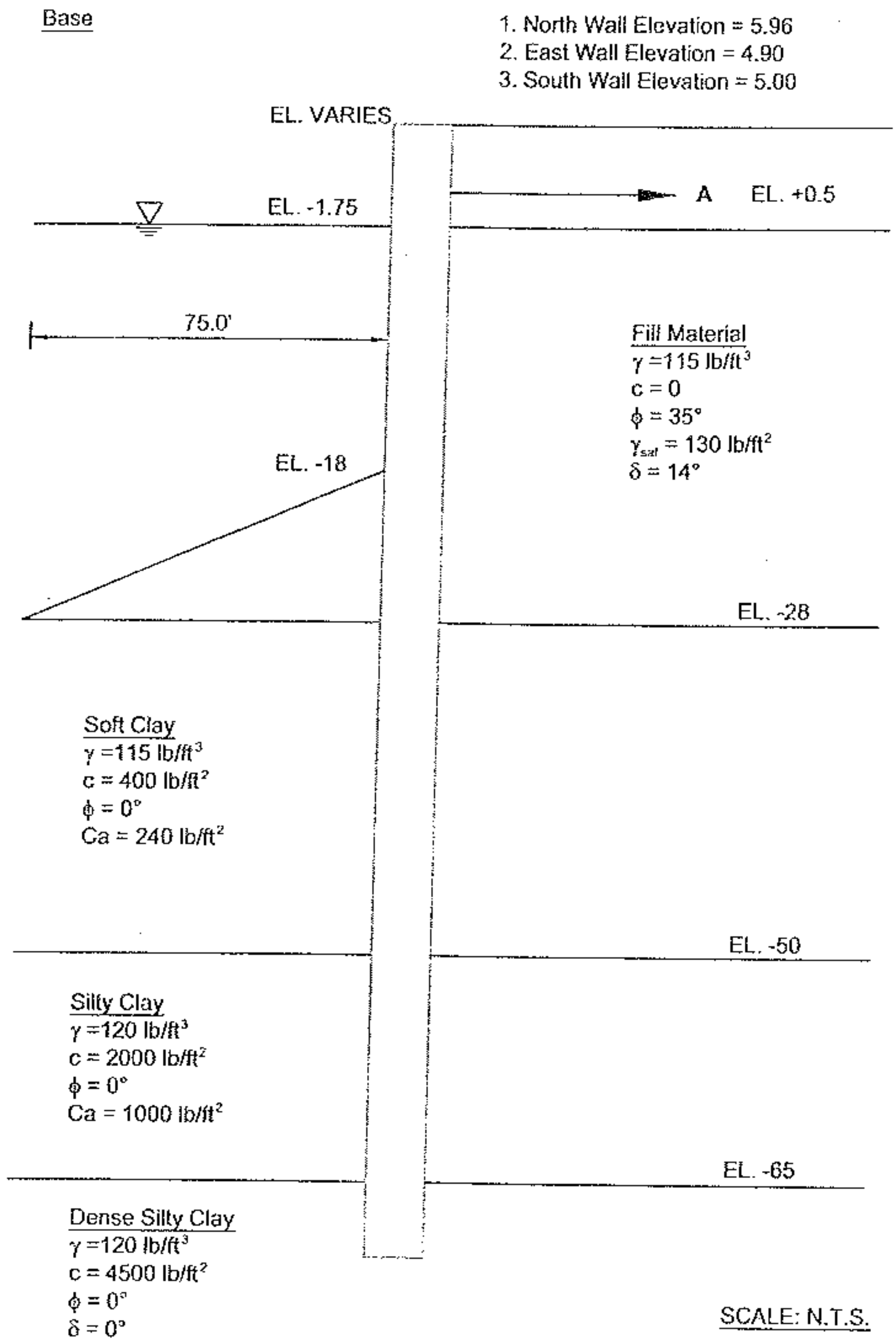
EXHIBIT 6
Typical Wall Patching Detail



DUSABLE PARK DEVELOPMENT
RIVER WALL CONDITION ASSESSMENT
FEBRUARY 2001

HARZA

EXHIBIT 7
Engineering Evaluation
Baseline Conditions



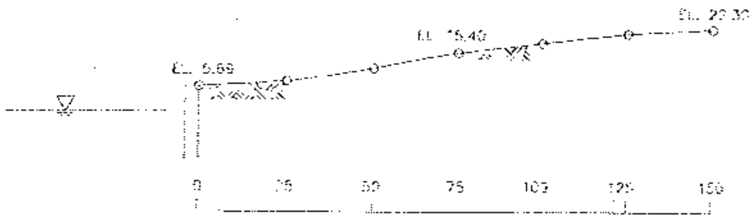
SCALE: N.T.S.

HARZA

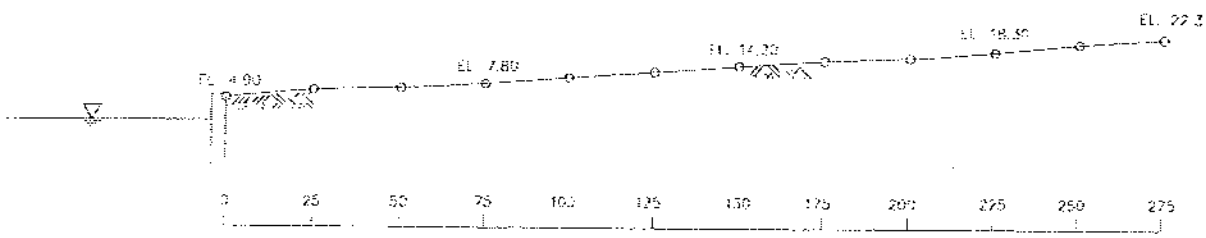
DUSABLE PARK DEVELOPMENT
RIVER WALL CONDITION ASSESSMENT
FEBRUARY 2001

EXHIBIT 8
Engineering Evaluation
Existing Conditions

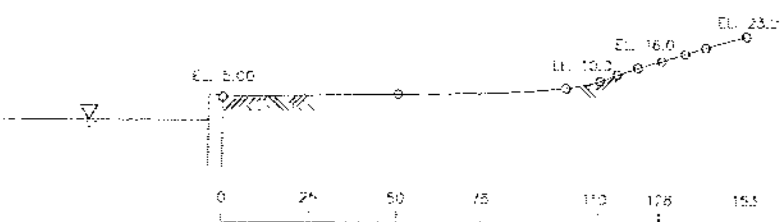
TYPICAL CROSS SECTIONS



NORTH WALL ELEVATION
SCALE: N.T.S.



EAST WALL ELEVATION
SCALE: N.T.S.

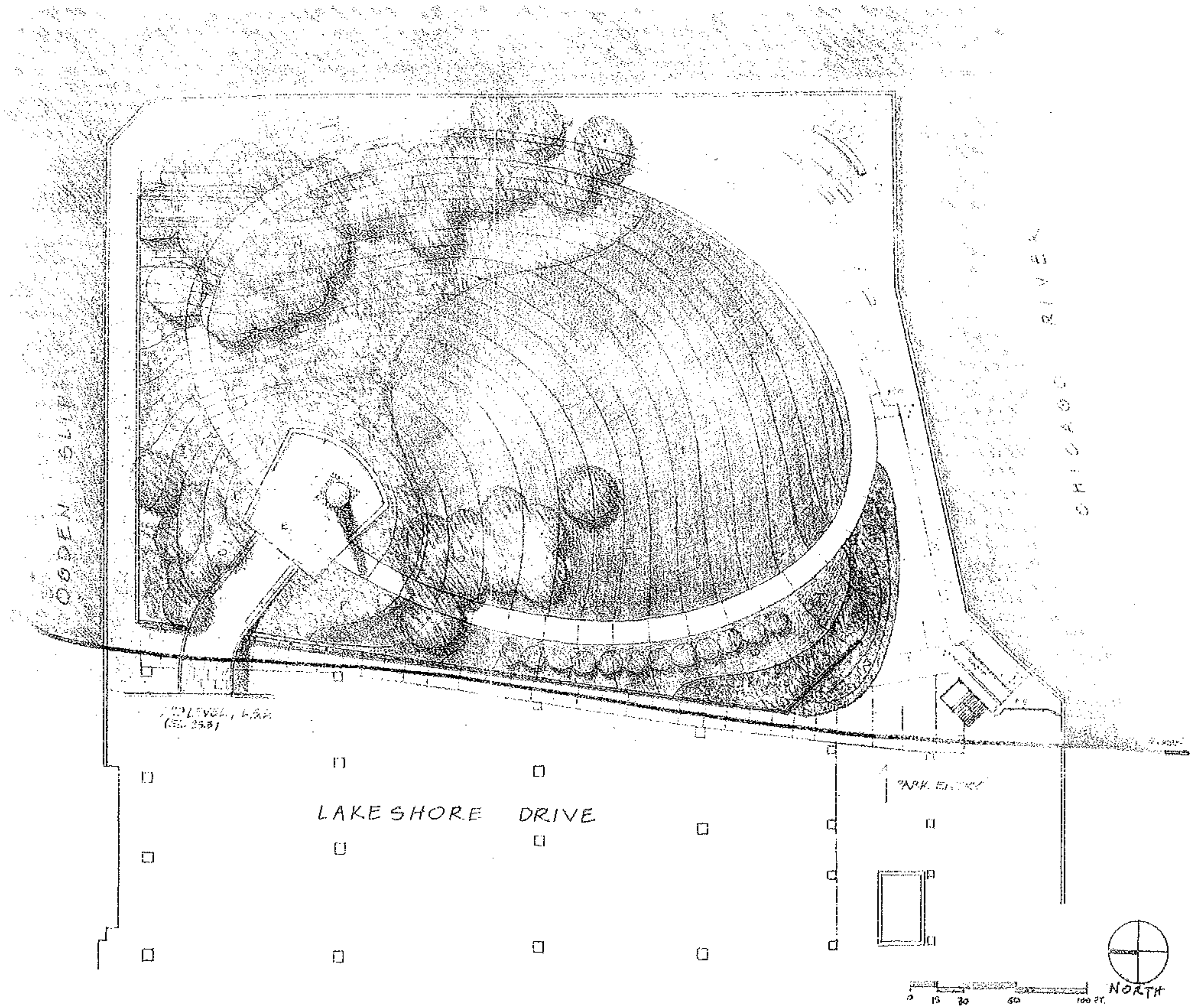


SOUTH WALL ELEVATION
SCALE: N.T.S.

DUSABLE PARK DEVELOPMENT
RIVER WALL CONDITION ASSESSMENT
FEBRUARY 2001

HARZA

EXHIBIT 9 PARK DEVELOPMENT (PLAN VIEW)
(COURTESY CHICAGO PARK DISTRICT)



CHICAGO PARK DISTRICT

REVISIONS DATE

APPROVED

PROJ. MGR.:
DESIGNER:
DRAWN:
CHECKED:
SCALE:
DATE:
SPEC. NO.:
JOB NO.

SHEET INFORMATION

PARK NO. / PROJECT NO.

EXHIBIT 10 PARK DEVELOPMENT (SECTIONS)
(COURTESY CHICAGO PARK DISTRICT)

DUSABLE PARK · WEST-EAST ELEVATION

DUSABLE PARK · NORTH-SOUTH ELEVATION

CHICAGO PARK DISTRICT
425 East McFetridge Drive
Chicago, Illinois
60605

REVISIONS DATE
APPROVED
PROJ. MGR.
DESIGNER
DRAWN
CHECKED
SCALE
DATE
SPEC NO.
JOB NO.
SHEET INFORMATION
CONCEPTUAL PLAN
DUSABLE PARK
APRIL 22, 1999
PARK NO. / PROJECT NO.

APPENDIX A – COST ESTIMATES

DuSable Park River Wall Condition Assessment

Cost Estimate for Repairs

Item No.	Description	Quantity	Unit	Unit Cost	Cost
1	Mobilization/Demobilization	1	LS	$15,000	$15,000
2	Temporary Facilities	1	LS	$10,000	$10,000
3	Demolition	1	LS	$5,000	$5,000
4	Clearing and Grubbing	33000	SF	$0.15	$4,950
5	Excavation	2400	CY	$12	$28,800
6	Backfilling	1700	CY	$15	$25,500
7	Sheet Pile Removal	1800	SF	$20	$36,000
8	Steel Sheet Piles	3000	SF	$24	$72,000
9	Vertical HP Piles	400	LF	$40	$16,000
10	Batter HP Piles	940	LF	$45	$42,300
11	Wales	100	LF	$50	$5,000
12	Wall Patching	130	SF	$35	$4,550
13	Channel Cap	1100	LF	$32	$35,200
				Total	$300,300
				Total w/25% contingency	$375,000

NOTES:

1. Unit costs are estimated based on landside construction. Water based construction may significantly increase the costs.

2. Cost estimate assumes that all work will be perfromed as one contract at the same time.

3. Cost estimate is based on the assumption that there is no net increase in loading on the wall.

DuSable Park River Wall Condition Assessment

Cost Estimate for Entire Wall Replacement

Item No.	Description	Quantity	Unit	Unit Cost	Cost
1	Mobilization/Demobilization	1	LS	$50,000	$50,000
2	Temporary Facilities	1	LS	$50,000	$50,000
3	Demolition	1	LS	$50,000	$50,000
4	Clearing and Grubbing	33000	SF	$0.15	$4,950
5	Excavation	4000	CY	$12	$48,000
6	Backfilling	2400	CY	$15	$36,000
7	Sheet Pile Removal	5500	SF	$20	$110,000
8	Steel Sheet Piles	49500	SF	$24	$1,188,000
9	Vertical HP Piles	320	LF	$40	$12,800
10	Batter HP Piles	8000	LF	$45	$360,000
11	Wales	1100	LF	$50	$55,000
12	Wall Patching	0	SF	$35	$0
13	Channel Cap	1100	LF	$32	$35,200
				Total	$1,999,950
				Total w/25% contingency	$2,500,000

NOTES:

1. Unit costs are estimated based on landside construction. Water based construction may significantly increase the costs.

2. Cost estimate assumes that all work will be perfromed as one contract at the same time.

3. Cost estimate is based on the assumption that there is no net increase in loading on the wall.

APPENDIX B – LIFE-CYCLE COST ANALYSIS

DuSable Park River Wall

Estimated Costs (2001) for Various Repair Rehabilitation & Operation and Maintenance Activities

	Minimum Repairs Alternative					Entire New Wall Alternative				
Year	Visual Insp.	Diver Insp.	Patching	Wall Repair & Replacement	Total	Visual Insp.	Diver Insp.	Patching	Wall Repair & Replacement	Total
	Initial Construction Cost Estimate (w/contingency)				$375,000					$2,500,000
1	$7,500				$7,500	$1,920				$1,920
2	$7,500	$5,000	$25,000		$37,500					$0
3	$7,500				$7,500	$1,920				$1,920
4	$7,500	$5,000	$25,000		$37,500					$0
5	$7,500			$375,000	$382,500	$1,920	$5,000			$6,920
6	$7,500	$5,000	$25,000		$37,500					$0
7	$7,500				$7,500	$1,920				$1,920
8	$7,500	$5,000	$25,000		$37,500					$0
9	$7,500				$7,500	$1,920				$1,920
10	$7,500	$5,000		$1,250,000	$1,262,500		$5,000			$5,000
11	$7,500				$7,500	$1,920				$1,920
12	$7,500	$5,000	$25,000		$37,500					$0
13	$7,500				$7,500	$1,920				$1,920
14	$7,500	$5,000	$25,000		$37,500					$0
15	$7,500			$375,000	$382,500	$1,920	$5,000			$6,920
16	$7,500	$5,000	$25,000		$37,500					$0
17	$7,500				$7,500	$1,920				$1,920
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20				$2,500,000	$2,500,000		$5,000			$5,000
21	$1,920				$1,920	$1,920				$1,920
22					$0					$0
23	$1,920				$1,920	$1,920				$1,920
24					$0					$0
25	$1,920	$5,000			$6,920	$1,920	$5,000			$6,920
26					$0					$0
27	$1,920				$1,920	$1,920				$1,920
28					$0					$0
29	$1,920				$1,920	$1,920				$1,920
30		$5,000			$5,000				$375,000	$375,000
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32					$0	$7,500	$5,000	$25,000		$37,500
33	$1,920				$1,920	$7,500				$7,500
34					$0	$7,500	$5,000	$25,000		$37,500
35	$1,920	$5,000			$6,920	$7,500			$375,000	$382,500
36					$0	$7,500	$5,000	$25,000		$37,500
37	$1,920				$1,920	$7,500				$7,500
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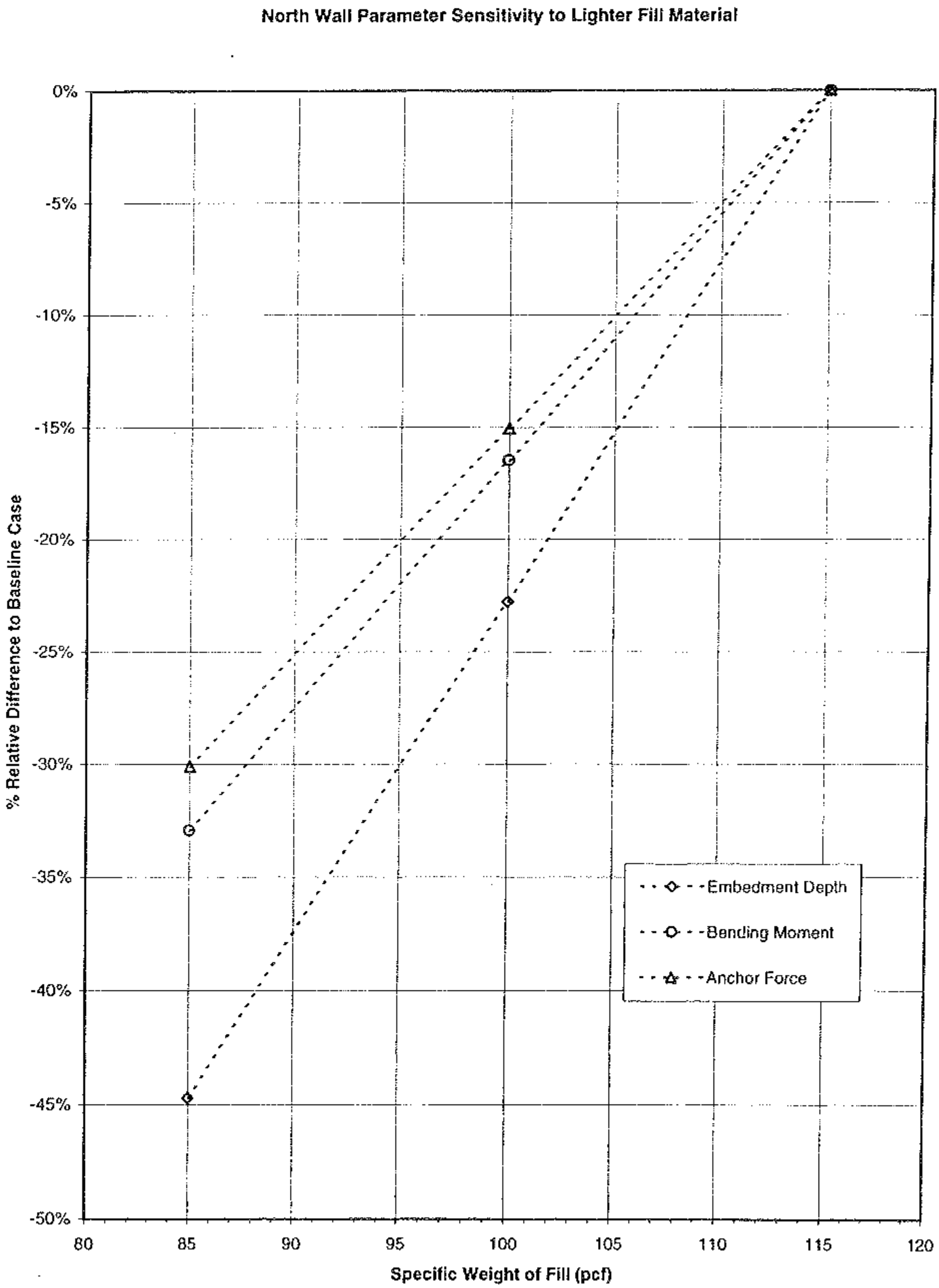
39	$1,920				$1,920	$7,500				$7,500
40		$5,000			$5,000	$7,500	$5,000		$1,250,000	$1,262,500
41	$1,920				$1,920	$7,500				$7,500
42					$0	$7,500	$5,000	$25,000		$37,500
43	$1,920				$1,920	$7,500				$7,500
44					$0	$7,500	$5,000	$25,000		$37,500
45	$1,920				$1,920	$7,500			$375,000	$382,500
46					$0	$7,500	$5,000	$25,000		$37,500
47	$1,920				$1,920	$7,500				$7,500
48					$0	$7,500	$5,000	$25,000		$37,500
49	$1,920				$1,920	$7,500				$7,500
50				$375,000	$375,000	$7,500	$5,000	$25,000	$375,000	$412,500
Net Present Worth (6%)					$2,544,362					$2,849,778
Net Present Worth (8%)					$2,071,614					$2,680,472
Net Present Worth (12%)					$1,470,467					$2,556,110

APPENDIX C – ENGINEERING EVALUATION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-10%
-15%
-20%
-25%
-30%
-35%
-40%
-45%
-50%
% Relative Difference to Baseline Case
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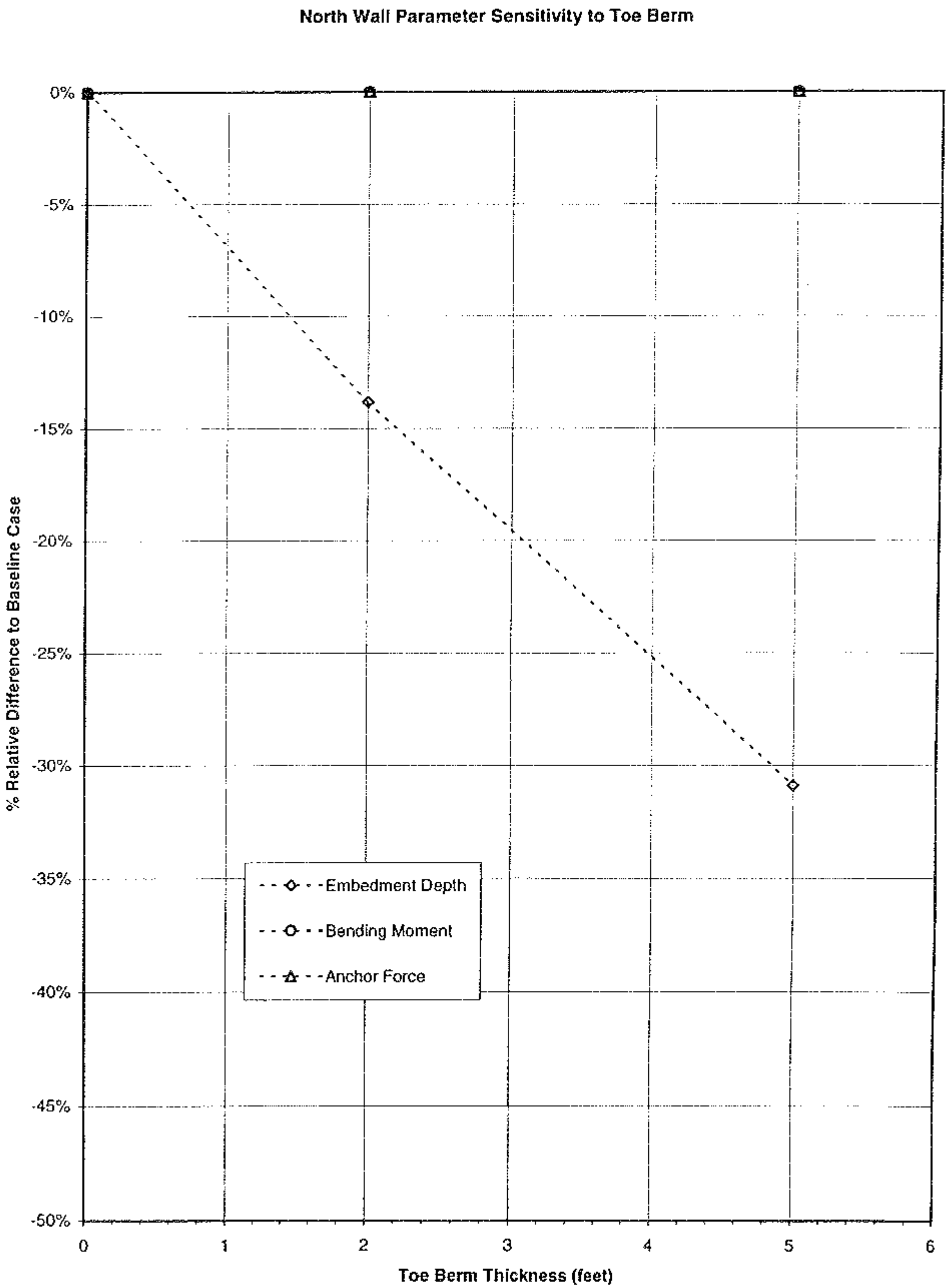
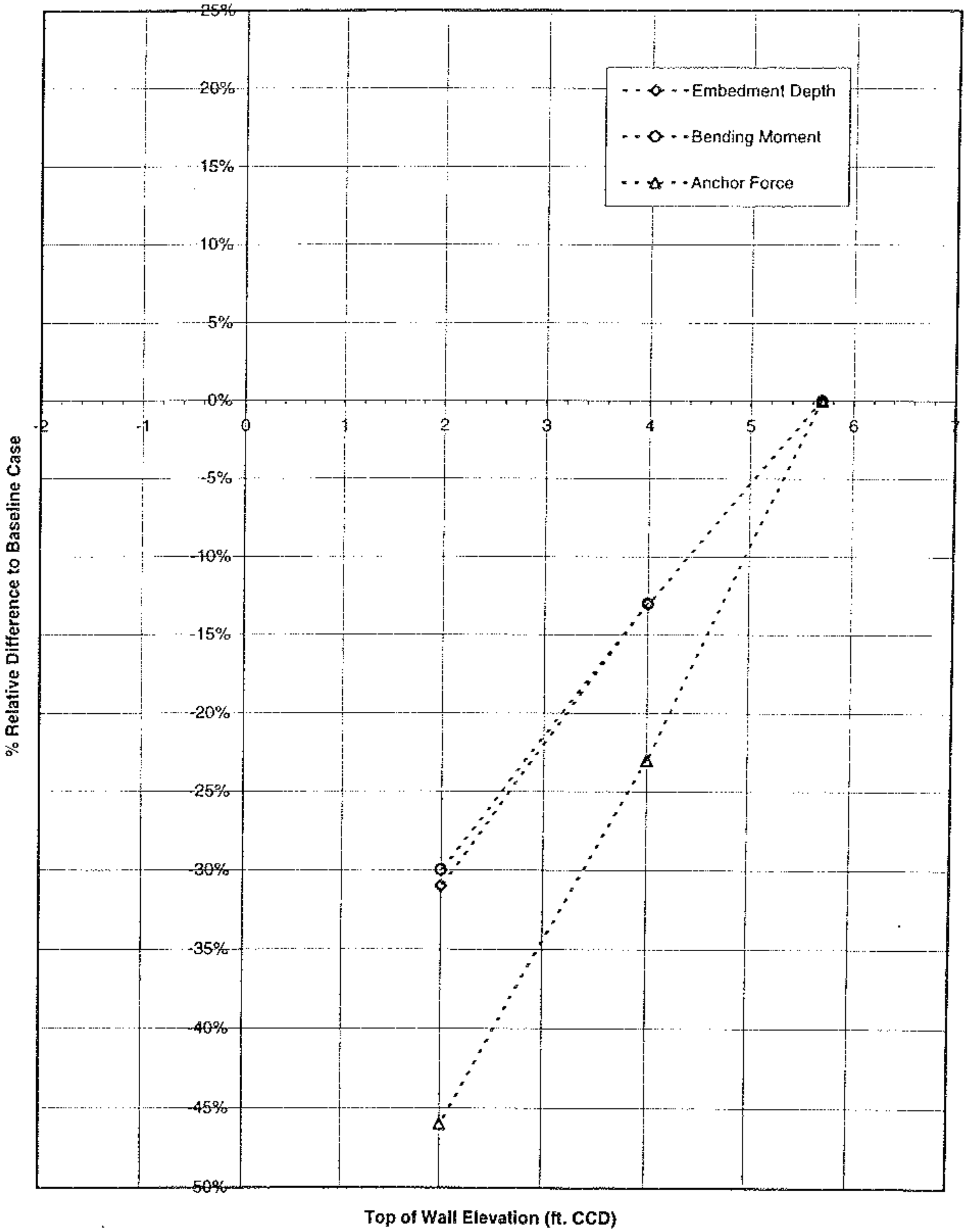
85
90
95
100
105
110
115
120
Specific Weight of Fill (pcf)
Embedment Depth
Bending Moment
Anchor Force


North Wall Parameter Sensitivity to Toe Berm
% Relative Difference to Baseline Case
0%
-5%
-10%
-15%
-20%
-25%
-30%
-35%
-40%
-45%
-50%
0
1
2
3
4
5
6
Toe Berm Thickness (feet)
Embedment Depth
Bending Moment
Anchor Force

HARZA

North Wall Parameter Sensitivity to Top of Wall Elevation



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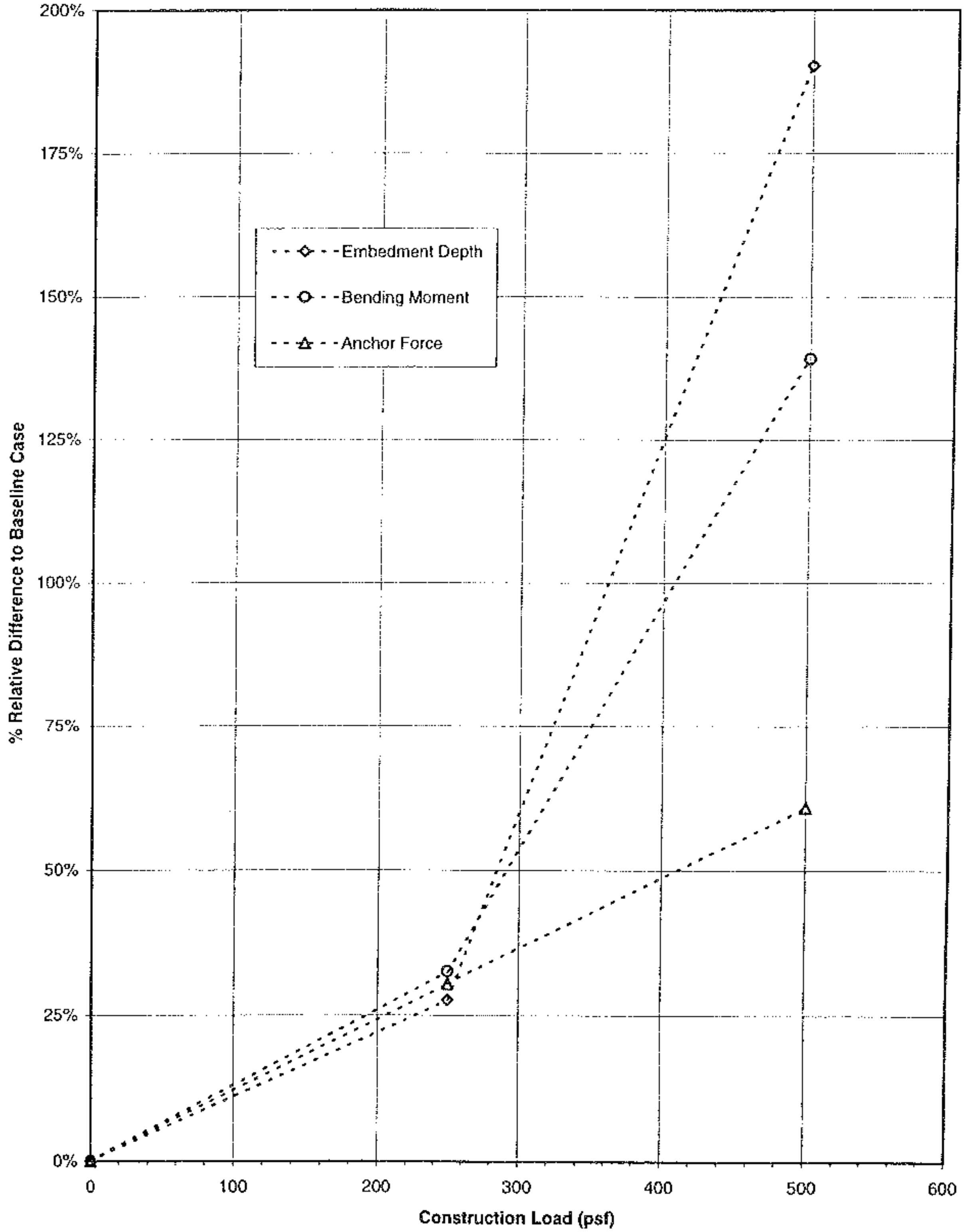
North Wall Parameter Sensitivity to Construction Load



HARZA

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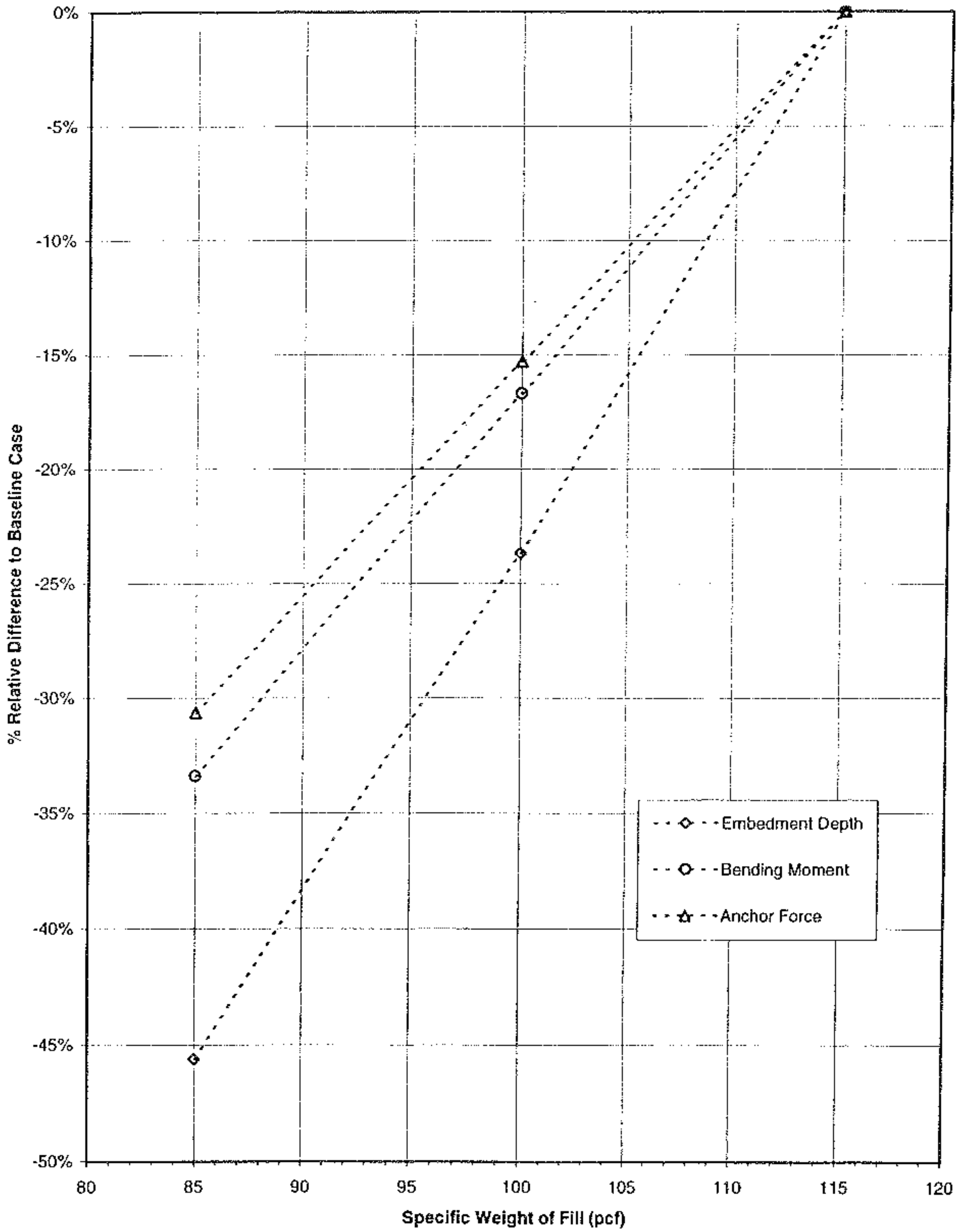

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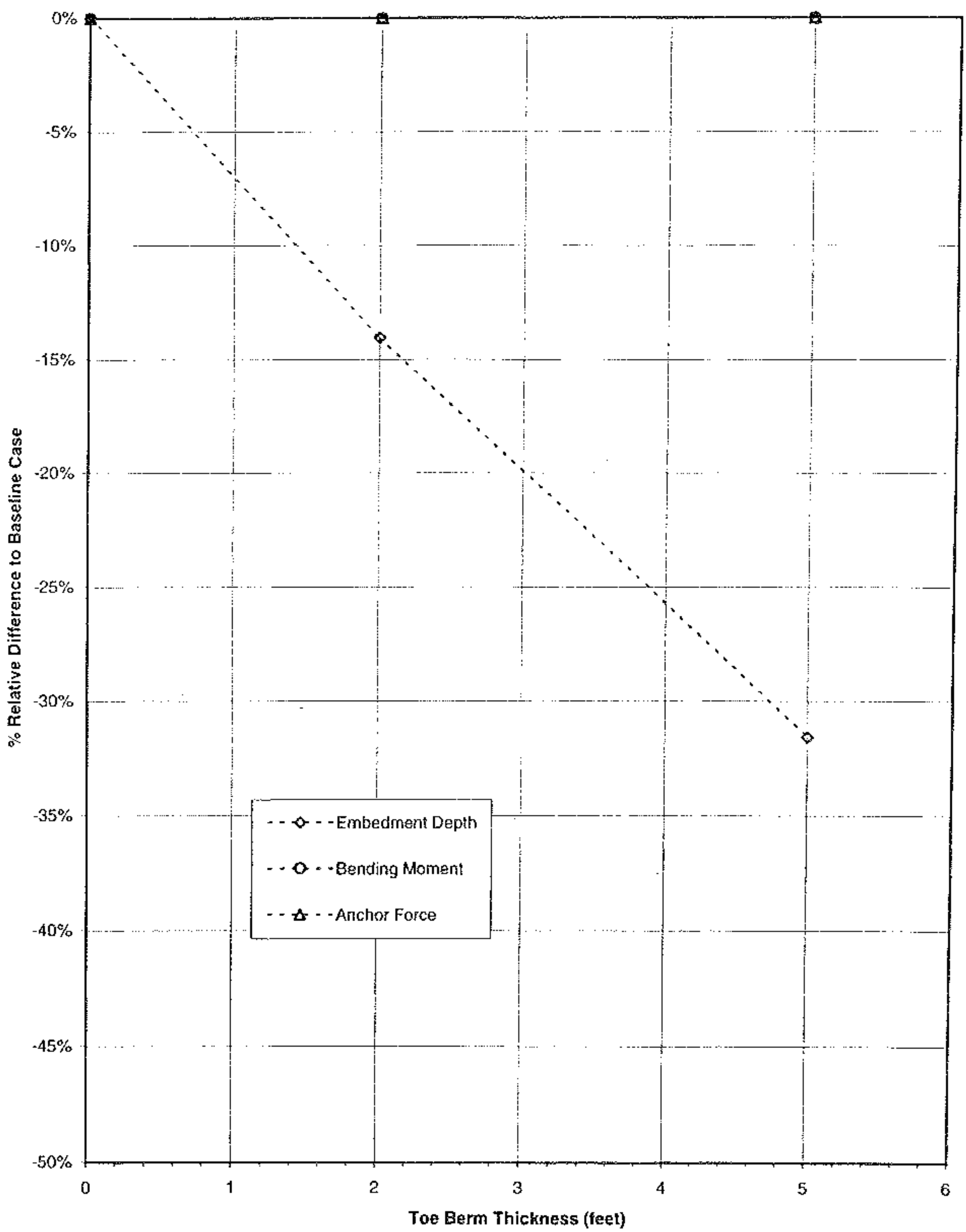

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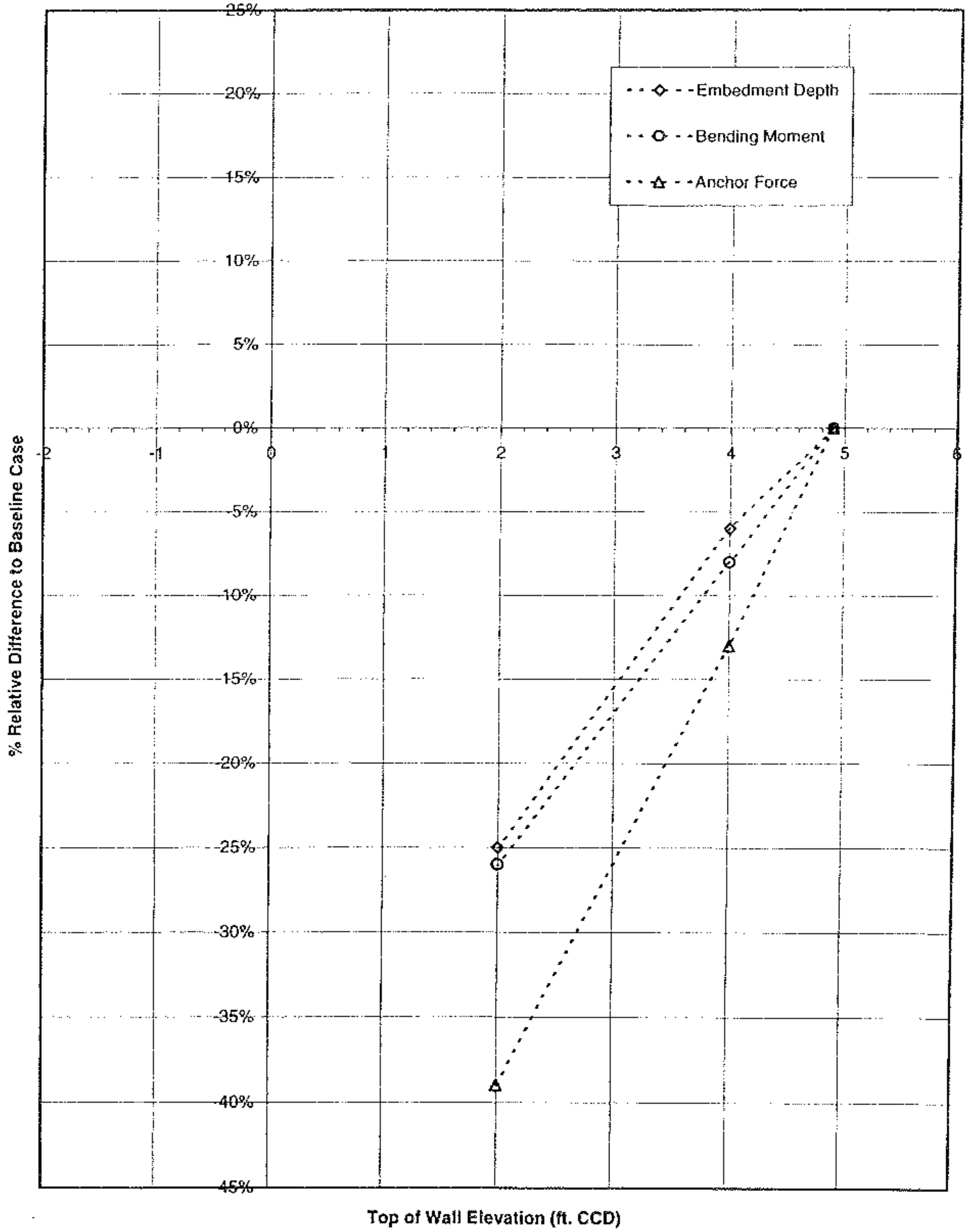
East Wall Parameter Sensitivity to Top of Wall Elevation



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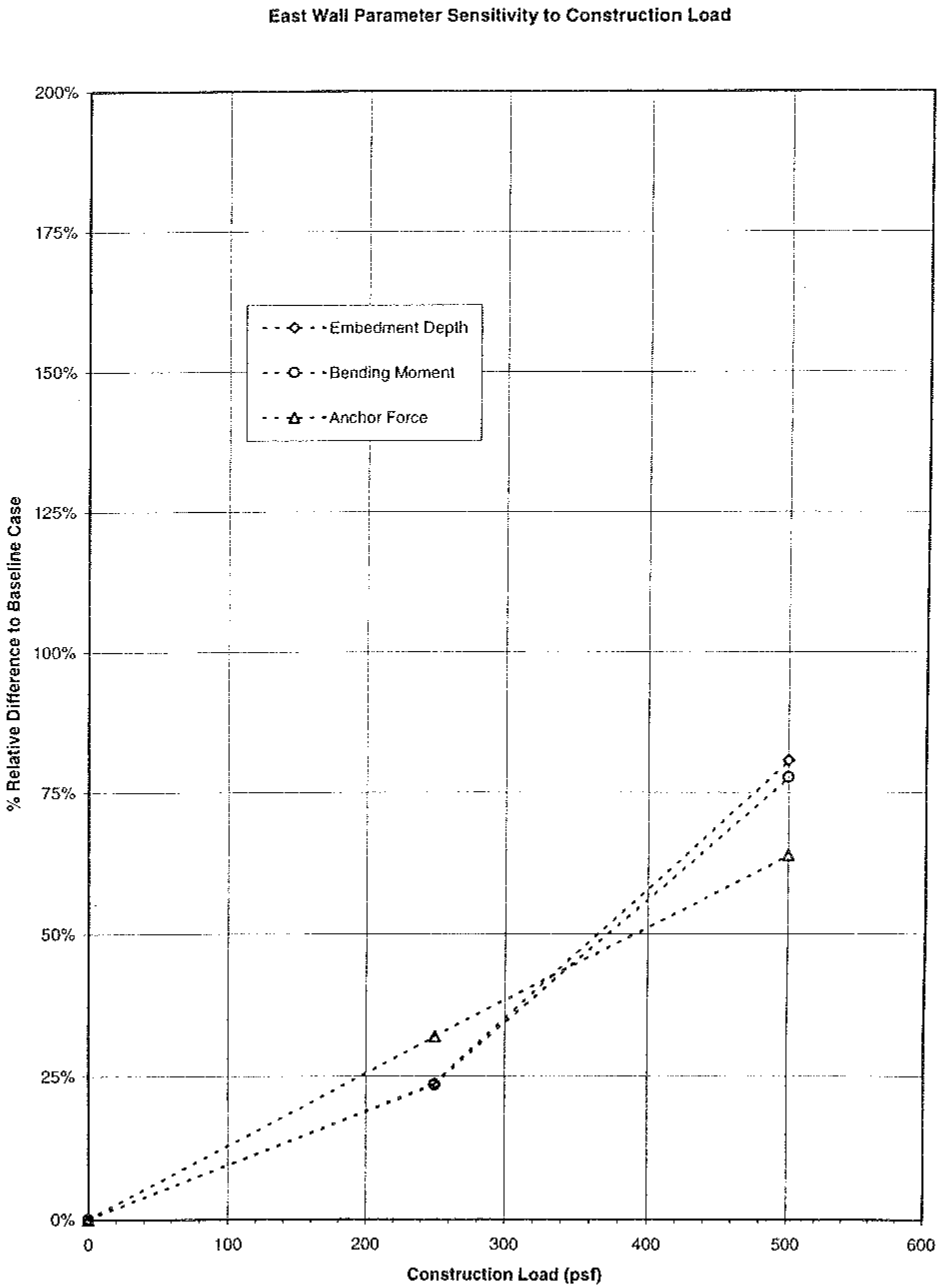
East Wall Parameter Sensitivity to Construction Load
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175%
150%
125%
100%
75%
50%
25%
0%
% Relative Difference to Baseline Case
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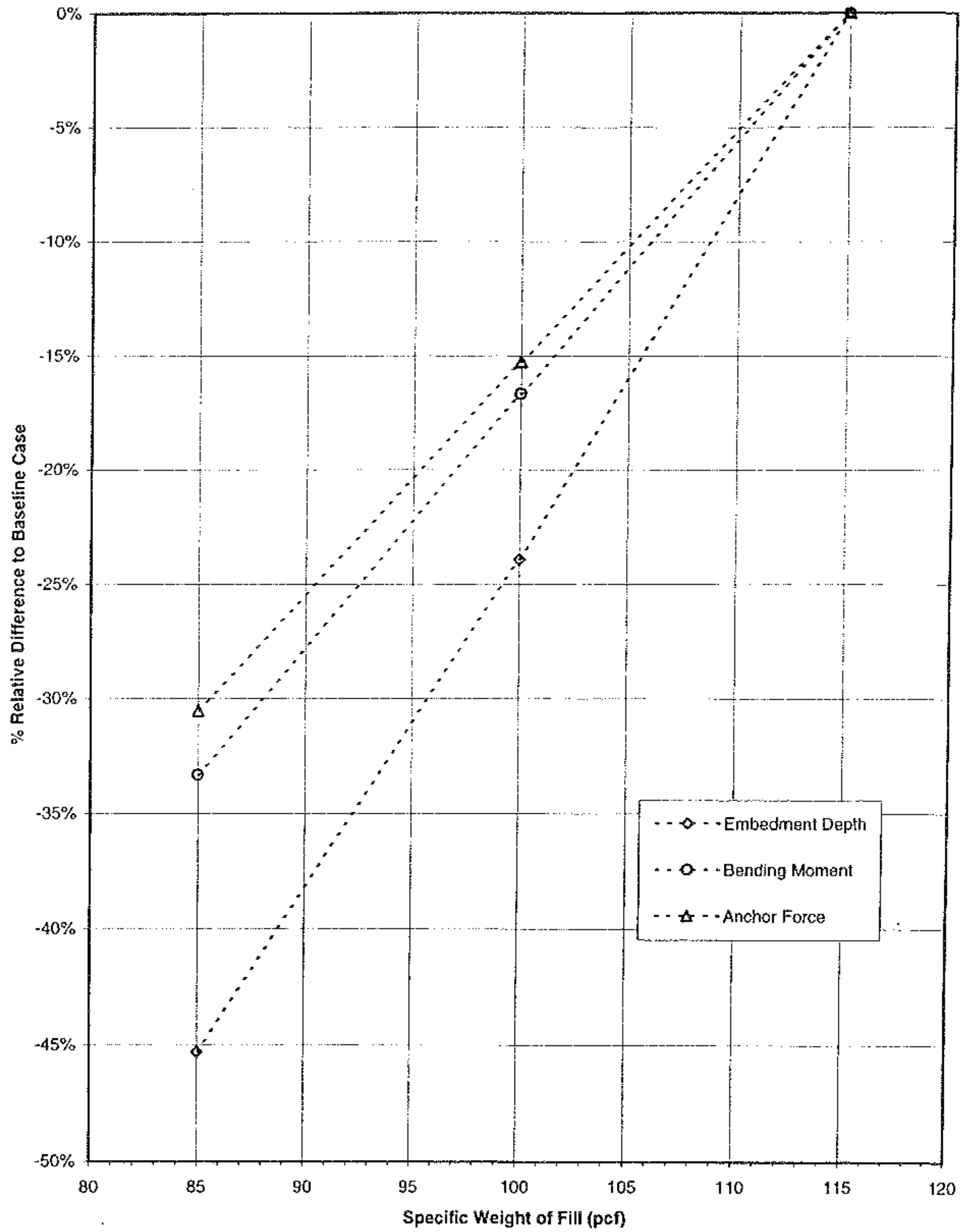
100
200
300
400
500
600
Construction Load (psf)
Embedment Depth
Bending Moment
Anchor Force

HARZA

South Wall Parameter Sensitivity to Lighter Fill Material



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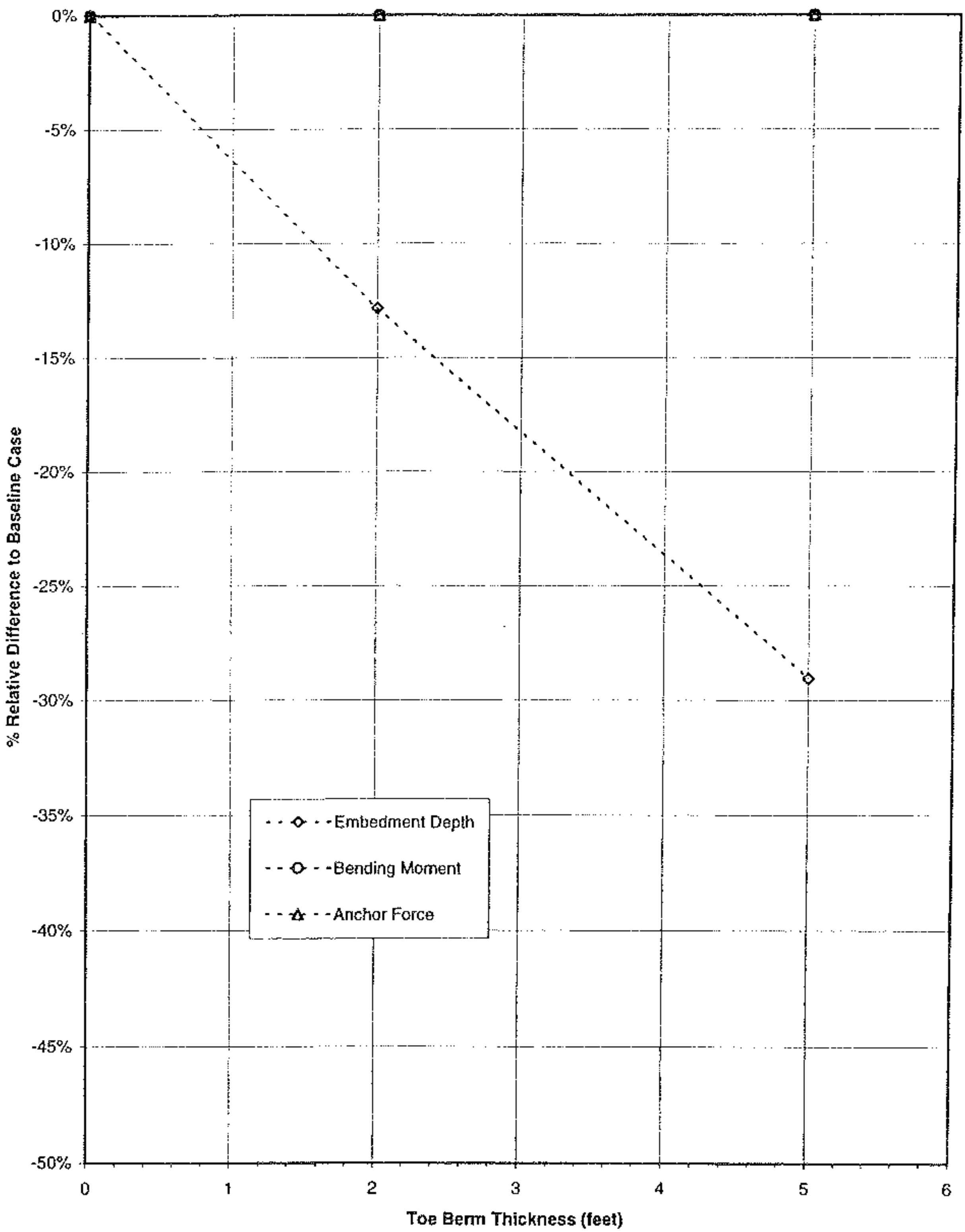
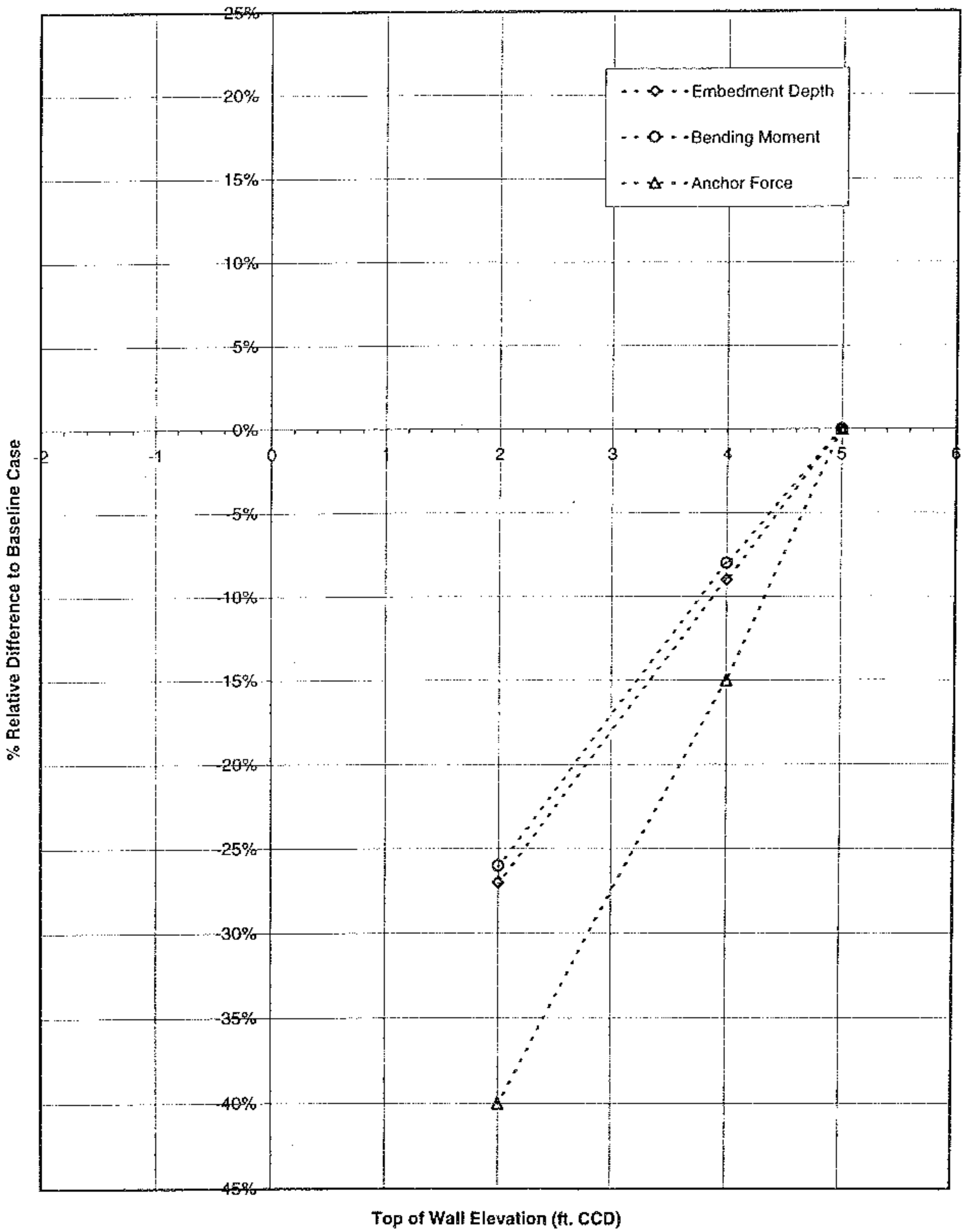
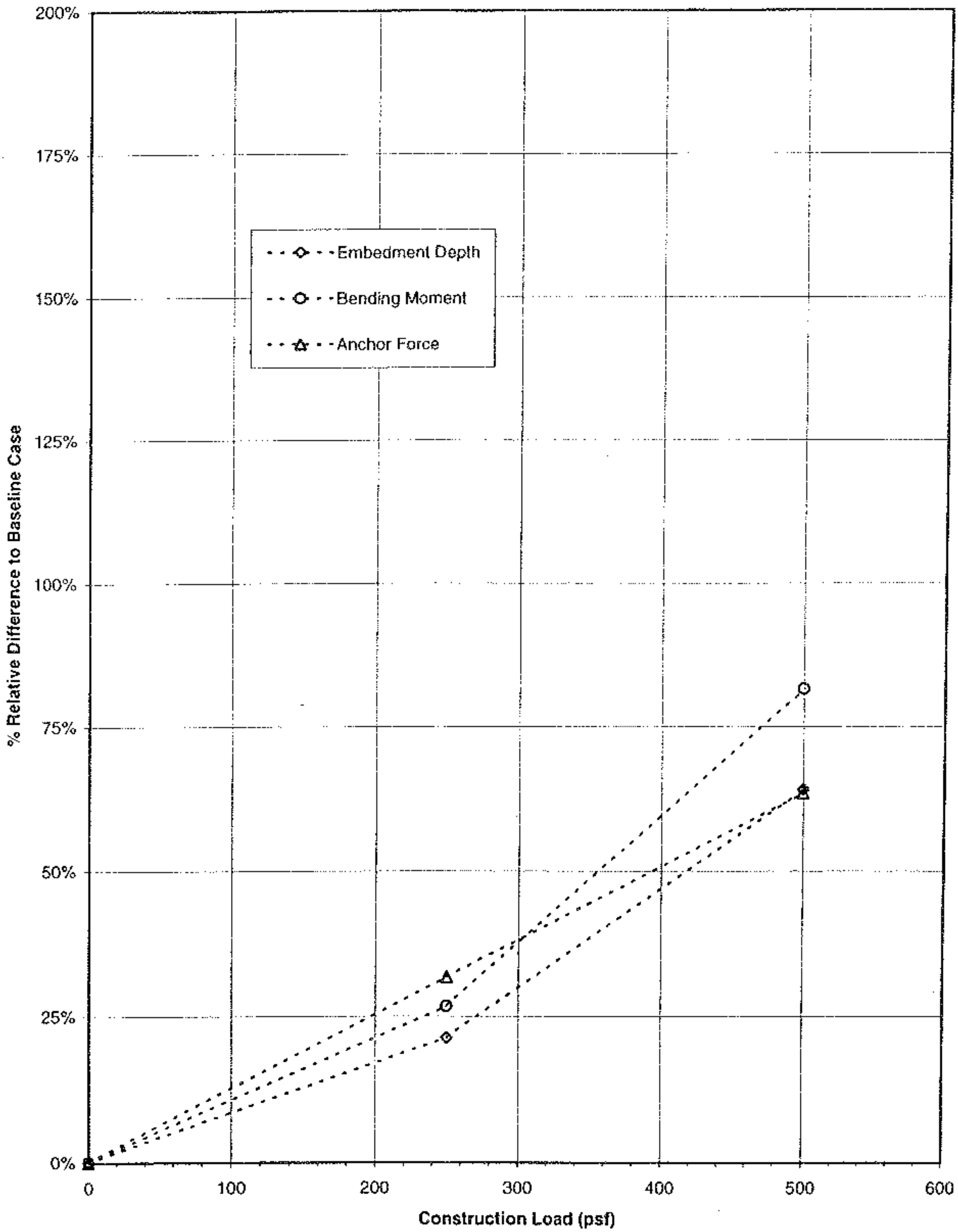


HARZA

South Wall Parameter Sensitivity to Top of Wall Elevation



South Wall Parameter Sensitivity to Construction Load



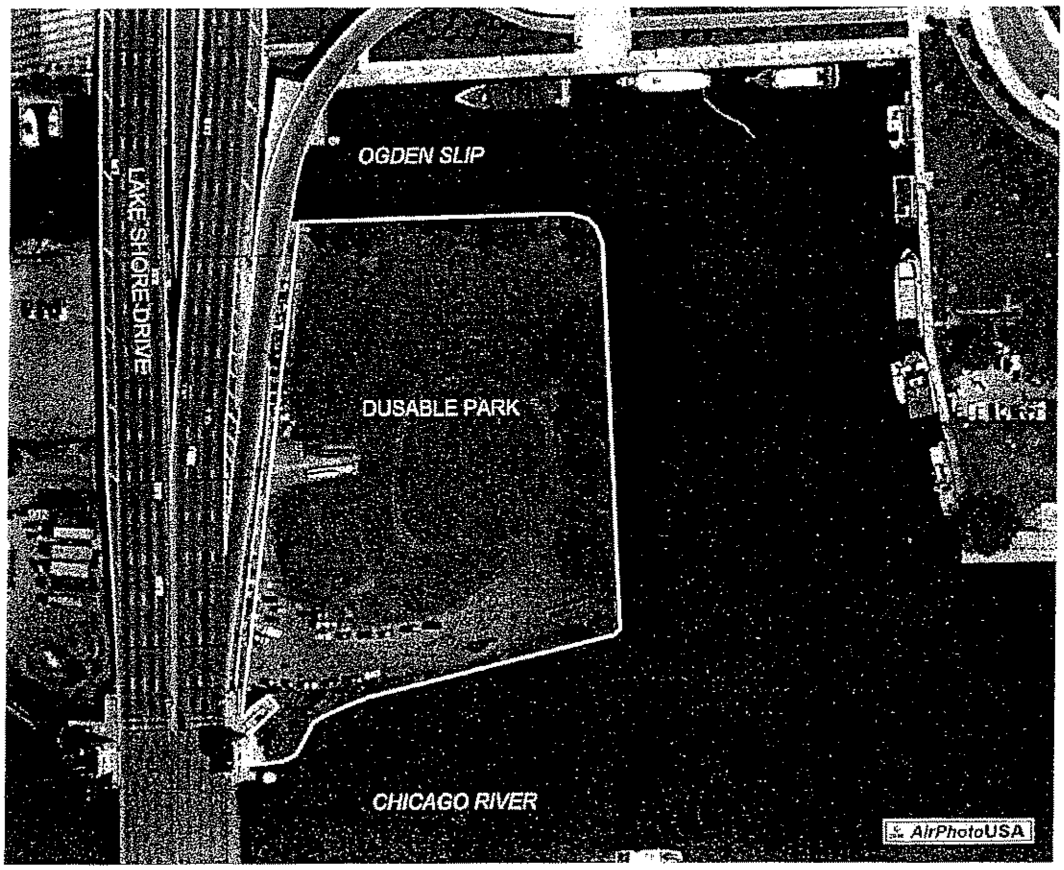
HARZA

Appendix H

Underwater Investigation of the DuSable Park Dockwall

COLLINS ENGINEERS INC.

UNDERWATER INVESTIGATION
OF THE
DUSABLE PARK DOCKWALL
ALONG THE
MAIN BRANCH OF THE CHICAGO RIVER
IN
CHICAGO, ILLINOIS



APRIL 2005

PREPARED FOR

KUDRNA & ASSOCIATES, LTD.

UNDERWATER INVESTIGATION

OF THE

DUSABLE PARK DOCKWALL

ALONG THE

MAIN BRANCH OF THE CHICAGO RIVER

IN

CHICAGO, ILLINOIS

APRIL 2005

Prepared For:

KUDRNA & ASSOCIATES, LTD.

Prepared by:

COLLINS ENGINEERS, INC.
123 North Wacker Drive, Suite 300
Chicago, IL 60606

COLLINS JOB NO. 4372

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

UNDERWATER INVESTIGATION

OF THE

DUSABLE PARK DOCKWALL

ALONG THE

MAIN BRANCH OF THE CHICAGO RIVER

IN

CHICAGO, ILLINOIS

1.0 INTRODUCTION

1.1 Purpose and Scope

This report consists of the results of a detailed underwater investigation of DuSable Park Dockwall along the Main Branch of the Chicago River in Chicago, Illinois.

Collins Engineers, Inc. conducted the underwater investigation for Kudrna & Associates, LTD. (Kudrna) on April 7, 2005. The work performed included a detailed inspection of the substructure components located in the water at the time of the investigation from the waterline to the channel bottom. In addition, a brief inspection was also made of those areas above the waterline that could be submerged during periods of higher water. Soundings of the channel bottom were taken along the face of the dockwall and 20 feet from the dockwall at 50-foot increments. Two excavations were also performed adjacent to the dockwall on July 19, 2005 to determine the condition and configuration of the wall anchorage system.

The following report includes a description of the structure, the method of investigation, a description of existing conditions, and an evaluation and recommendations based on the findings.

1.2 General Description of the Structure

DuSable Park is a 3.5-acre parcel of land owned by the Chicago Park District (Park District). The Park District is in the process of developing this unused parcel of land into a public park. The land in question is located east of Lake Shore Drive in Chicago, Illinois. The Ogden Slip and the Main Branch of the Chicago River provide the northern, eastern, and southern borders of the park. Refer to Figure 1 in Appendix A for a Location Map. The portion of the park adjacent to the waterway consists of 1125 linear feet of dockwall. Refer to Photographs 1 through 3 in Appendix B for overall views of the DuSable Park dockwall.

1.3 Method of Investigation

A detailed field inspection was conducted to determine the physical condition of the steel sheeting from the waterline to the channel bottom. A brief visual examination of the dockwall above the waterline was also made.

A four-person team, consisting of a licensed structural engineer-diver, two engineer-divers, and a technician-diver conducted the underwater inspection. During the inspection, the divers were able to work from a boat, where an engineer recorded the inspection notes. Scuba equipment was used to perform the underwater inspection, consisting of a visual and tactile examination of the entire surface of the dockwall from waterline to channel bottom, with particular attention given to any noted areas of excessive deterioration or apparent distress. Photographs were taken to document typical conditions and any deterioration. Several areas on the underwater surfaces of the dockwall were cleaned so that the condition could be more closely examined. Observations of the channel bottom adjacent to the dockwall were also made. The type of channel bottom material, presence and location of scour holes, presence or absence of riprap, and the presence of debris was noted.

The location of the waterline with respect to the dockwall was noted and water depth soundings were taken with a Fathometer along the dockwall perimeter. A sounding plan was developed using these soundings. Refer to Figure 2 in Appendix A for the sounding plan along the dockwall.

2.0 EXISTING CONDITIONS

At the time of the inspection, the waterline of the Main Branch of the Chicago River was located approximately 7.0 feet below the top of the dockwall at Station 2+00. This corresponds to a waterline elevation of −2.07 feet Chicago City Datum (CCD), based on USGS data taken at Columbus Drive. Refer to Figure 2 in Appendix A for the dockwall configuration and sounding plan.

Around the perimeter of the dockwall, the channel bottom material typically consisted of silty sand and random interspersed construction debris, with up to 1.5 feet of probe rod penetration. Refer to Figures 3 and 4 in Appendix A for the Dockwall Plan and Inspection Notes.

<u>Station 0+00 to 0+60</u>

The dockwall in this area consisted of timber Wakefield sheeting with a concrete cap. Timber piles, measuring approximately 12 inches in diameter, were located approximately 1 foot in front of the timber sheeting. The outer layer of timber sheeting was in satisfactory condition with 1/8-inch awl penetrations and random 2-inch wide gaps between sheets. Interior timber piles filled in the gaps at all observed locations. Above water, the concrete cap was typically in fair condition with heavy concrete scale along the bottom corner, having up to 4 inches of penetration. This scale extended 18 inches along the vertical cap face and 12 inches along the cap underside. Random reinforcement was observed in this area, having up to 15 percent loss of section. The protective timber fender was in satisfactory condition with light weathering and random 1/2-inch wide checking. Below water, there was a 1-inch thick layer of marine and aquatic growth extending from the waterline to the channel bottom. Refer to Photographs 4 and 5 in Appendix B for views of the dockwall in this area.

<u>Station 0+60 to 0+70</u>

The dockwall in this area consisted of steel sheeting, with a concrete cap. Timber piles, measuring approximately 12 inches in diameter, were located in front of the steel sheeting at 2.5-foot centers. Between the sheeting and piles were timber stringers measuring 8 inches by 12 inches which acted as spacers. The stringers were located along the mudline and 4 feet above the channel bottom. Above water, the concrete cap was typically in fair condition with heavy concrete scale along the bottom corner, having up to 4 inches of penetration. This scale extended 18 inches along the vertical cap face and 12 inches along the cap underside. Random reinforcement was observed in this area, having up to 15 percent loss of section. The protective timber fender was in satisfactory condition with light weathering and random 1/2-inch wide checking.

Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet, with up to 10 percent loss of section. In addition, there was a 1-inch thick layer of marine and aquatic growth extending from the waterline to the channel bottom.

<u>Station 0+70 to 5+85</u>

The dockwall in this area was constructed of steel sheeting. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet with up to 10 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing. Heavy impact damage was observed from Station 2+63 to Station 2+80, extending from 1 foot below the waterline to the top of the wall. All interlocks were intact, except for one location at Station 2+67. This interlock had up to 1 inch of separation from 3 feet below the top of the sheeting to the waterline. In addition, a 1-inch thick layer of marine and aquatic growth extended from the waterline to the channel bottom. Refer to Photographs 6 through 10 in Appendix B for views of the dockwall in this area.

Above water, random minor areas having up to 100 percent loss of section were observed, typically measuring 2 inches in diameter with a maximum area of 8 inches by 8 inches. Additionally, random areas of impact damage extended along the top 6 inches of the dockwall from Station 0+70 to Station 3+25. The steel had indentations measuring up to 6 inches deep with random small areas having up to 100 percent loss of section.

Between Station 3+25 and Station 3+66, the frequency of the missing fender anchors increased creating a 3-inch diameter hole in every other outer sheet face. These holes were typically located between 2 feet and 3 feet above the waterline. The steel sheeting in this area also exhibited random burn holes, measuring 3 inches in diameter.

The interlocks along the waterline typically exhibited up to 30 percent section loss between Station 3+35 and Station 3+66. In this area, the steel sheeting exhibited moderate impact damage causing tears along the faces of the steel sheeting and up to 50 percent loss of section. Additionally, no tie rods were visible along this section of wall.

From Station 3+66 to Station 5+25, approximately 75 percent of the fender anchors were missing. Between Station 5+25 and Station 5+85, approximately 20 percent of the fender anchors were missing.

Station 5+85 to 7+75

The dockwall in this area consisted of steel sheeting. The steel plate washers located on every other outer pan face had failed or were heavily corroded in locations where the threaded anchor rod extended outward. At locations where the anchor heads were located along the exterior wall face, the washers typically exhibited light to moderate corrosion. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet with up to 10 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing. All interlocks were intact, with a 1-inch thick layer of marine and aquatic growth extending from the waterline to the channel bottom. Refer to Photograph 11 in Appendix B for a view of the dockwall in this area.

Station 7+75 to 9+25

The dockwall in this area was constructed of steel sheeting. Along this portion of the wall, heavy pack rust was observed between the plate washers and sheeting. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/16-inch deep pitting over 25 percent of the steel surface area and at the interlocks. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet with up to 10 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing. All interlocks were intact, with a 1-inch thick layer of marine and aquatic growth extending from the waterline to the channel bottom. Refer to Photograph 12 for a view of the dockwall in this area.

Station 9+25 to 10+85

The dockwall in this area consisted of steel sheeting. Along this portion of the wall, the anchor rod nuts typically exhibited up to 25 percent section loss, with random nuts exhibiting up to 75 percent loss of section. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/16-inch deep pitting. Heavy pitting, measuring up to 1/8-inch deep, extended down 5 feet from the waterline. Above water, the sheeting typically exhibited heavy section loss from the waterline up 3 feet with 50 percent loss of section. The heaviest section loss was located at 3 feet above the waterline, where there was up to 100 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing. All interlocks were intact, with a 1-inch thick layer of marine and aquatic growth extending from the waterline to the channel bottom. Refer to Photographs 13 through 17 in Appendix B for views of the dockwall in this area.

Station 10+85 to 11+25

The dockwall in this area was constructed of steel sheeting. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet with up to 10 percent loss of section. Above water, the anchor washers typically exhibited up 10 percent section loss. All interlocks were intact, with a 1-inch thick layer of marine and aquatic growth extending from the waterline to the channel bottom. Refer to Photograph 18 for a view of the dockwall in this area.

3.0 EXCAVATION FINDINGS

Two areas of the retained soil adjacent to the dockwall were excavated on July 19, 2005. Excavations were performed at Station 3+94 and Station 9+50 to determine the condition and configuration of the sheeting anchorage system.

Station 3+94

The dockwall anchorage system in this area typically consisted of a 1-1/2 inch diameter steel rod located 5.5 feet below the top of the sheeting. A 3-foot long section of the rod was heavily corroded adjacent to the steel sheeting, with up to 75 percent loss of section. Further excavation of this area revealed that the rod extended approximately 35 feet from the dockwall. The western end of the rod was free, with no anchorage system observed. In addition, the interior face of the sheet pile wall was heavily corroded, with up to 20 percent loss of section. Refer to Figure 5 in Appendix A for a section view of the dockwall at Station 3+94. Refer to Photographs 19 through 21 in Appendix B for views of the excavation area.

While excavating this area, the northern end of the dockwall anchorage system was observed from Station 3+35 to Station 3+60. The anchorage system consisted of 1-1/2 inch diameter steel rods extending approximately 28 feet from the dockwall. The rods were anchored to timber railroad ties, measuring 12 inches by 12 inches. No additional wall anchorage components, such as sheeting or soldier piles, were observed. It should be noted that no ties extended through the dockwall in this area.

Station 9+50

The dockwall anchorage system in this area typically consisted of a 1-1/2 inch diameter steel rod located approximately 7 feet below the top of the sheeting. Light corrosion of the anchor was evident, with less than 10 percent loss of section. The rod extended approximately 3 feet behind the steel sheeting, where it was attached to two channels. However, no additional wall anchorage components were observed in this area. In addition, moderate oxidation of the interior face of the sheet pile wall was observed, having less than 10 percent loss of section. Refer to Figure 6 in Appendix A for a section view of the dockwall at Station 9+50. Refer to Photographs 22 and 23 in Appendix B for views of the excavation area.

4.0 EVALUATION AND RECOMMENDATIONS

Overall, the DuSable Park dockwall was generally in poor condition. The deterioration and damage to the steel sheet piling coupled with the lack of a structurally adequate anchorage system make the possibility of repairs cost prohibitive. Currently, portions of the steel sheet pile dockwall are acting as cantilevers, greatly reducing the structural integrity of the wall system.

Based on the underwater inspection findings and the excavation observations, it is recommended that the existing steel sheet pile dockwall be removed and replaced with a properly designed earth retention system. Refer to Figure 7 in Appendix A for a section view of a commonly used dockwall configuration. Regardless of how the replacement dockwall system is configured, the structure should be designed and sealed by a Licensed Structural Engineer in the state of Illinois.

Preliminary estimates indicate that the cost to remove the existing wall and replace it with a structurally adequate system, as depicted in Figure 7 of Appendix A, will be approximately \$5,710,000. This estimate includes the cost of removing the existing steel sheeting, furnishing and erecting new steel sheeting with a structural anchorage system, and installing new protective timber fenders. Refer to the spreadsheet located on the next page for a detailed cost estimate to remove and replace the existing dockwall.

Collins appreciates this opportunity to be of service to Kudrna with regard to this dockwall assessment. Please note that we have considerable experience in all phases of the design and management of new dockwall construction, and would like to assist you in that regard, if and when the need arises. If you have any questions regarding this report, please contact me at 312.704.9300.

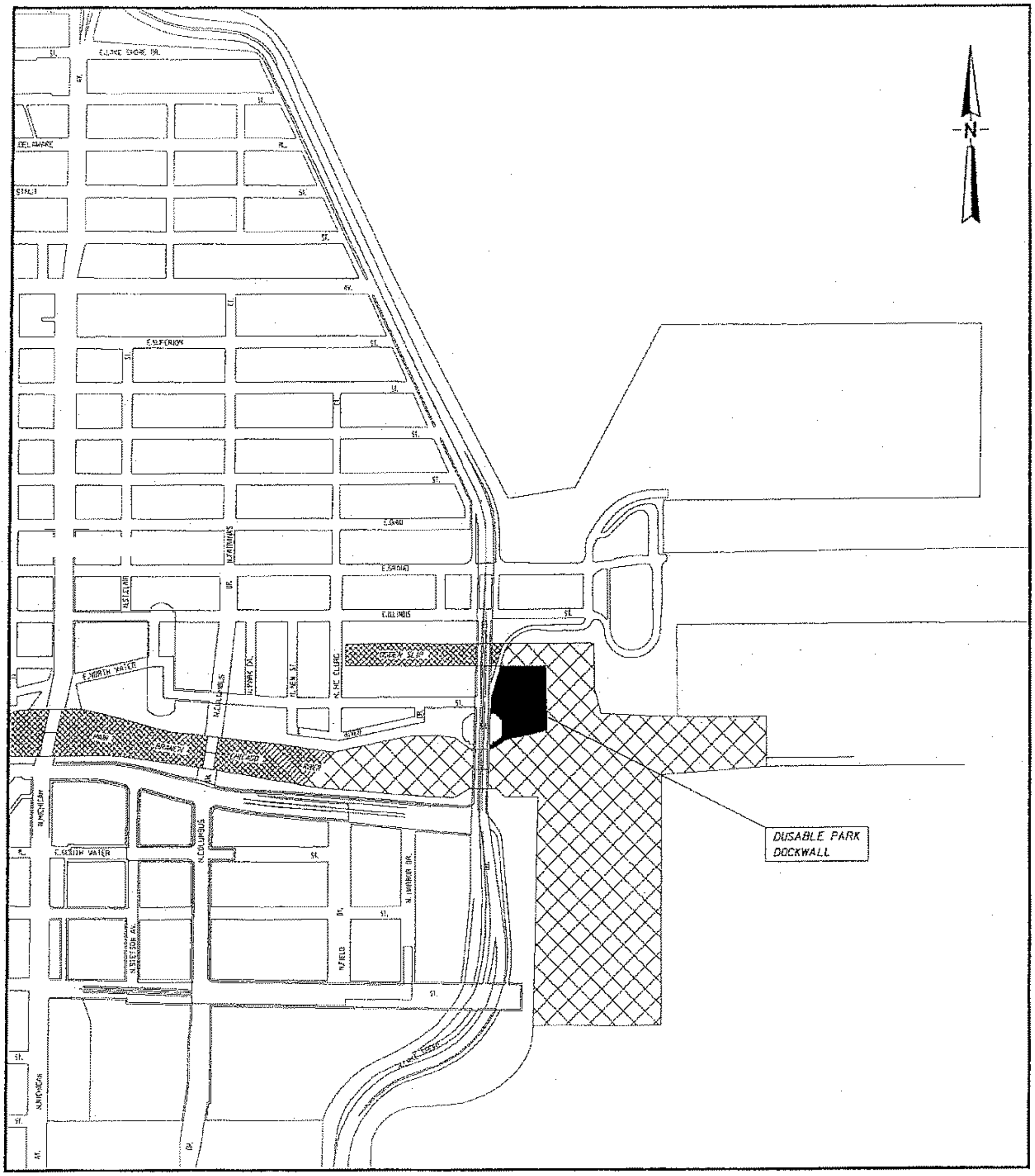
0804.05
Exp. 11.30.06

Respectfully submitted,

COLLINS ENGINEERS, INC.

John E. O'Leary, P.E., S.E.

DuSable Park Dockwall						
Item No.	Category	Pay Item Description	Unit	Quantity	Unit Price	Total
1	S	REMOVAL OF EXISTING SHEETING	L SUM	1	$400,000	$400,000
2	S	FURNISHING AND ERECTING STRUCTURAL STEEL	L SUM	1	$1,592,340	$1,592,340
3	S	FURNISHING STEEL PILES HP14X73	LIN FT	21400	$48	$1,027,200
4	S	STEEL SHEET PILING	SQ FT	69700	$38	$2,648,600
5	S	TIMBER FENDER SYSTEM	LIN FT	1125	$35	$39,375
					TOTAL	**$5,707,515**



PARK #478
DUSABLE PARK

DUSABLE PARK DOCKWALL

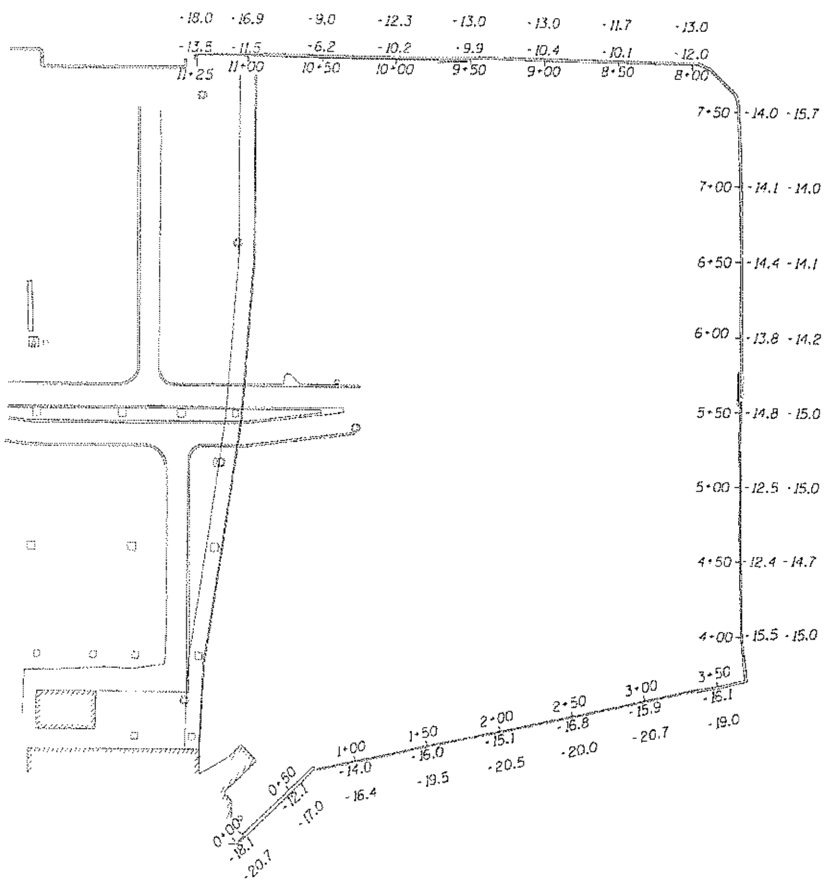
LOCATION MAP

Drawn By: DR	COLLINS ENGINEERS INC. 123 North Wacker Drive Suite 300 Chicago, IL 60606 (312) 704-9300 www.collinsengr.com	Date: AUGUST, 2005
Checked By: JEO	ILLINOIS PROFESSIONAL DESIGN FIRM LICENSE NO. 184-000993	Scale: NONE
Code: 43720001		Figure No.: 1

FILE: g:\4372\43720001.dgn

Appendix A

Figures



General Notes:

1. Approximately 1125 linear feet of the DuSable Park dockwall was inspected under water.

2. At the time of the inspection on April 7, 2005 the waterline was located approximately 7.0 feet below the top of the dockwall at Station 2+00. This corresponds to a waterline elevation of -2.07 feet Chicago City Datum (CCD), based on USGS data taken at Columbus Drive.

3. Soundings indicate the channel bottom depths at the time of inspection and are measured in feet.

4. Soundings were taken parallel to the dockwall, as well as 20 feet from the dockwall.

LEGEND:

-14.0 Channel Bottom Depth

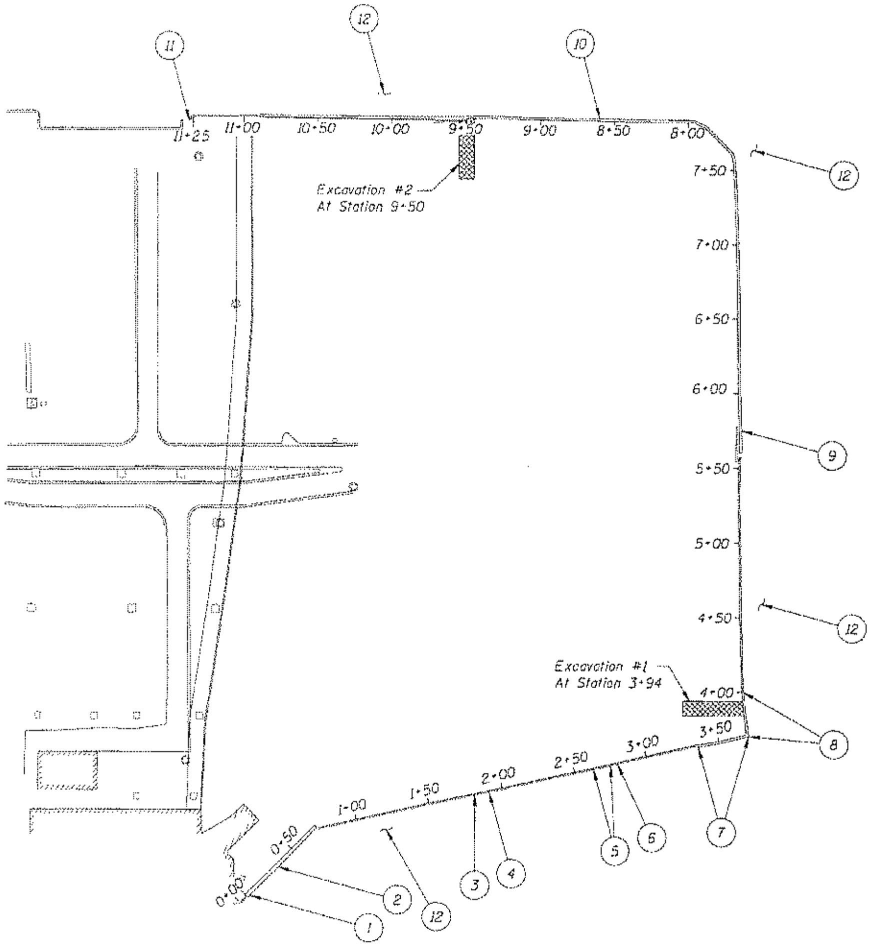
PARK #478
DUSABLE PARK

DUSABLE PARK DOCKWALL

SOUNDING PLAN

Drawn By: DR	COLLINS ENGINEERS	Date: AUGUST, 2005
Checked By: JEC		Scale: 1"=80'
Code: 43720001		Figure No: 7

FILE: g:\4372\43720001.dgn



Inspection Notes:

(1) Concrete section loss along east side of expansion joint from the waterline up 2 feet. Area measured 1 foot in width with up to 18 inches of penetration and exposed steel reinforcement having up to 15 percent loss of section.

(2) Concrete section loss along construction joint from the waterline up 2 feet. Area measured 2 feet in width with up to 8 inches of penetration.

(3) Area of 100 percent steel section loss measuring 16 inches in height and 8 inches in width at 2 feet above the waterline.

(4) Heavy impact damage extending downward 6.5 feet from the top of the steel sheeting, with an indentation measuring up to 1 foot deep and a 1 foot long tear in the sheeting.

(5) Heavy impact damage from 1 foot below the waterline to the top of the steel sheeting, with 1 foot deep indentations. The interlock at Station 2+67 had opened up to 1 inch wide from the waterline to 3 feet below the top of the steel sheeting. Vertical tears in the sheeting extended from 1 foot above the waterline to 4 feet below the waterline, with a 1 foot square area of 100 percent section loss at the waterline.

(6) Heavy impact damage from the top of the steel sheeting to 1 foot above the waterline. The steel sheeting exhibited 100 percent section loss in this area, with a visible loss of backfill material creating a 3-foot deep sinkhole behind the wall.

(7) No tie rods visible between Station 3+35 and Station 3+66.

(8) Heavy impact damage to steel channel whaler at 1.5 feet above the waterline from Station 3+56 to Station 4+00.

(9) Area of 100 percent steel section loss measuring 2.5 feet high and 2 inches wide, located 3 feet above the waterline.

(10) Outfall with hole in sheeting measuring 2 feet high by 3 feet wide at 4 feet above the waterline.

(11) Gap between steel sheeting and concrete dockwall, measuring up to 3 inches wide above the waterline and 18 inches wide below the waterline.

(12) The Channel bottom material typically consisted of silty sand and random interspersed construction debris, with up to 1.5 feet of probe rod penetration.

General Notes:

1. Refer to Figure 4 for General Dockwall Inspection Notes.

2. At the time of the inspection on April 7, 2005 the waterline was located approximately 7.0 feet below the top of the dockwall at Station 2+00. This corresponds to a waterline elevation of -2.07 feet Chicago City Datum (CCD), based on the USGS data taken at Columbus Drive..

3. Soundings indicate the channel bottom depths at the time of inspection and are measured in feet.

PARK #478
DUSABLE PARK

DUSABLE PARK DOCKWALL

PLAN AND INSPECTION NOTES

Drawn By: DR	COLLINS ENGINEERS	Date: AUGUST, 2005
Checked By: JEO		Scale: 1"=80'
Code: 4372000I		Figure No. 3

FILE: g:\4372\4372000I.dgn

General Dockwall Inspection Notes:

Station 0+00 to 0+60

• The dockwall in this area consisted of timber Wakefield sheeting with a concrete cap. The outer layer of timber sheeting was in satisfactory condition with 1/8-inch awl penetrations and random 2-inch wide gaps between sheets. Interior timber piles filled in the gaps at all observed locations. Above water, the concrete cap was typically in fair condition with heavy concrete scale along the bottom corner, having up to 4 inches of penetration. This scale extended 18 inches along the vertical cap face and 12 inches along the cap underside. Random reinforcement was observed in this area, having up to 15 percent loss of section. The protective timber fender was in satisfactory condition with light weathering and random 1/2-inch wide checking.

Station 0+60 to 0+70

• The dockwall in this area consisted of steel sheeting, with a concrete cap. Above water, the concrete cap was typically in fair condition with heavy concrete scale along the bottom corner, having up to 4 inches of penetration. This scale extended 18 inches along the vertical cap face and 12 inches along the cap underside. Random reinforcement was observed in this area, having up to 15 percent loss of section. The protective timber fender was in satisfactory condition with light weathering and random 1/2-inch wide checking. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet, with up to 10 percent loss of section.

Station 0+70 to 5+85

• The dockwall in this area was constructed of steel sheeting. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet, with up to 10 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing. Heavy impact damage was observed from Station 2+63 to Station 2+80, extending from 1 foot below the waterline to the top of the wall. All interlocks were intact, except for one location at Station 2+67. This interlock had up to 1 inch of separation from 3 feet below the top of the sheeting to the waterline.

• Above water, random minor areas having up to 100 percent loss of section were observed, typically measuring 2 inches in diameter with a maximum area measuring 8 inches by 8 inches. Additionally, random areas of impact damage extended along the top 6 inches of the dockwall from Station 0+70 to Station 3+25. The steel had indentations measuring up to 6 inches with random small areas having up to 100 percent loss of section.

• Between Station 3+25 and Station 3+66, the frequency of the missing fender anchors increased creating a 3-inch diameter hole in every other outer sheet face. These holes were typically located between 2 feet and 3 feet above the waterline. The steel sheeting in this area also exhibited random burn holes, also measuring 3 inches in diameter.

• The interlocks along the waterline typically exhibited up to 30 percent section loss between Station 3+35 and Station 3+66. In this area, the steel sheeting exhibited moderate impact damage causing tears along the faces of the steel sheeting and up to 50 percent loss of section. Additionally, no tie rods were visible along this section of wall.

• From Station 3+66 to Station 5+25, approximately 75 percent of the fender anchors were missing. Between Station 5+25 and Station 5+85, approximately 20 percent of the fender anchors were missing.

Station 5+85 to 7+75

• The dockwall in this area was constructed of steel sheeting. The steel plate washers located on every other outer pan face had failed or were heavily corroded in locations where the threaded anchor rod extended outward. At locations where the anchor heads were located along the exterior wall face, the washers typically exhibited light to moderate corrosion. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet, with up to 10 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing.

Station 7+75 to 9+25

• The dockwall in this area was constructed of steel sheeting. Along this portion of the wall, heavy pack rust was observed between the plate washers and sheeting. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/16-inch deep pitting over 25 percent of the steel surface area and at the interlocks. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet, with up to 10 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing.

Station 9+25 to 10+85

• The dockwall in this area was constructed of steel sheeting. Along this portion of the wall, the anchor rod nuts typically exhibited up to 25 percent section loss, with random nuts exhibiting up to 75 percent loss of section. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/16-inch deep pitting. Heavy pitting, measuring up to 1/8-inch deep, extended down 5 feet from the waterline. Above water, the sheeting typically exhibited heavy section loss from the waterline up 3 feet with 50 percent loss of section. The heaviest section loss was located at 3 feet above the waterline, where there was up to 100 percent loss of section. The timber fenders were typically missing and the remaining fender anchors were either deformed or missing.

Station 10+85 to 11+25

• The dockwall in this area was constructed of steel sheeting. Below water, the steel sheeting typically exhibited random rust nodules measuring up to 1 inch in diameter and 1/32-inch deep pitting over 25 percent of the steel surface area. A 1/16-inch thick layer of scale was also located on the sheeting surfaces below water. Heavier scale and pitting, measuring up to 1/8-inch deep, was located from the waterline down 2 feet, with up to 10 percent loss of section. Above water, the anchor washers typically exhibited up 10 percent section loss.

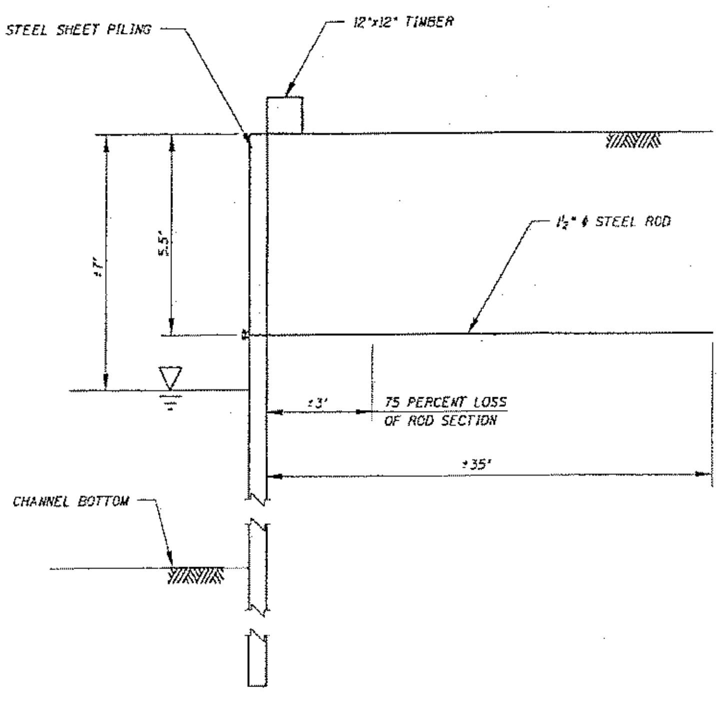
PARK #478
DUSABLE PARK

DUSABLE PARK DOCKWALL

GENERAL DOCKWALL INSPECTION NOTES

Drawn By: DR	COLLINS ENGINEERS	Date: AUGUST, 2005
Checked By: JEO		Scale: NTS
Code: 43720001		Figure No.: 4

FILE: g:\4372\4372000l.dgn



DOCKWALL SECTION AT STATION 3+94

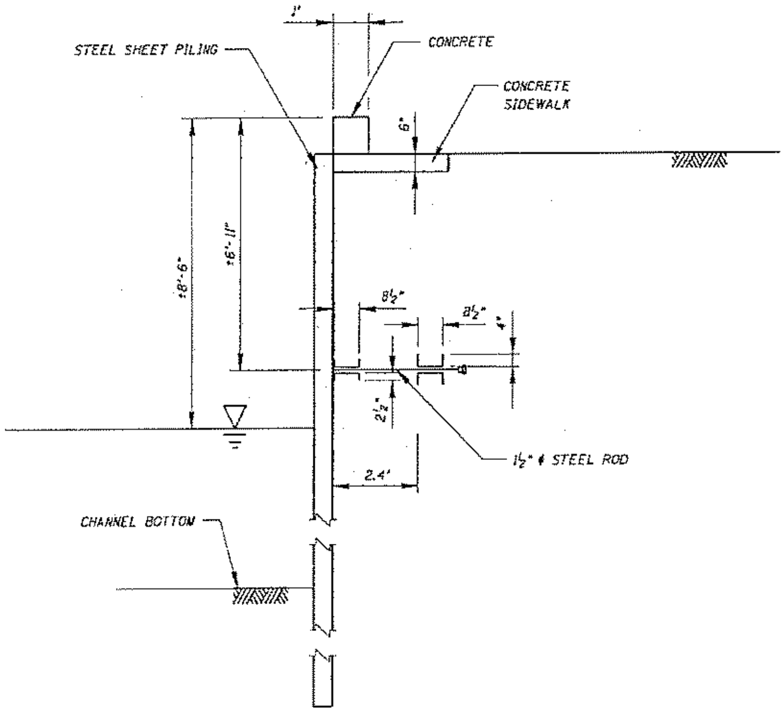
PARK #478
DUSABLE PARK

DUSABLE PARK DOCKWALL

EXISTING DOCKWALL SECTION
AT STATION 3+94

Drawn By: DR	COLLINS ENGINEERS INC. 123 North Wacker Drive Suite 300 Chicago, IL 60606 (312) 704-9300 www.collinsengr.com ILLINOIS PROFESSIONAL DESIGN FIRM LICENSE NO. 184-000592	Date: AUGUST, 2005
Checked By: JEO		Scale: NONE
Code: 43720001		Figure No.: 5

FILE: g:\4372\43720001.dgn



DOCKWALL SECTION AT STATION 9+50

PARK #478
DUSABLE PARK

DUSABLE PARK DOCKWALL

EXISTING DOCKWALL SECTION AT STATION 9+50

Drawn By: DR	COLLINS ENGINEERS INC. 123 North Wacker Drive Suite 300 Chicago, IL 60606 (312) 704-9300 www.collinsengr.com ILLINOIS PROFESSIONAL DESIGN FIRM LICENSE NO. 184-000993	Date: AUGUST, 2005
Checked By: JEO		Scale: NONE
Code: 43720001		Figure No.: 6

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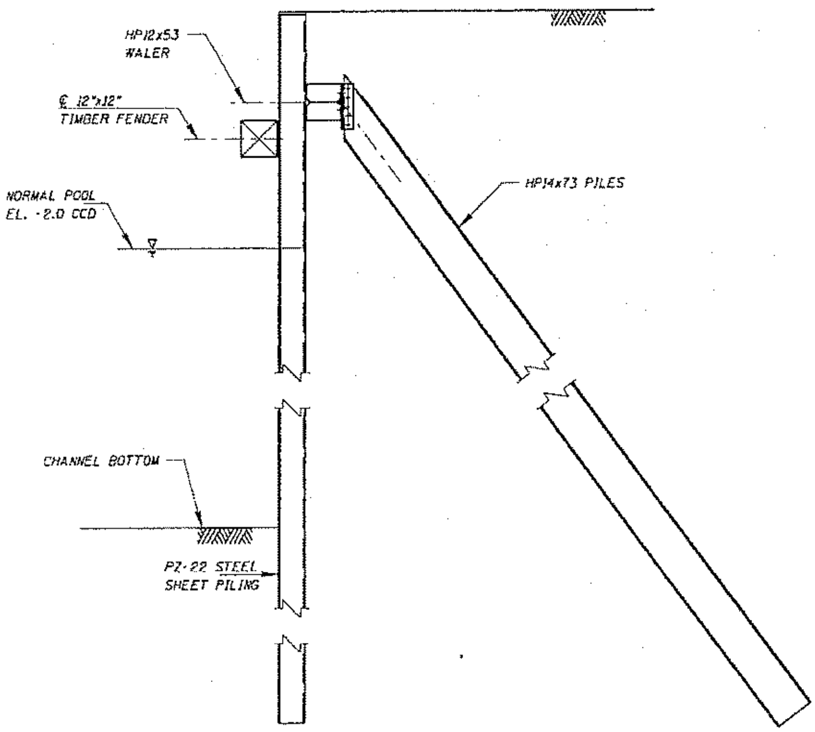


REPLACEMENT DOCKWALL SECTION

PARK #478
DUSABLE PARK

DUSABLE PARK DOCKWALL

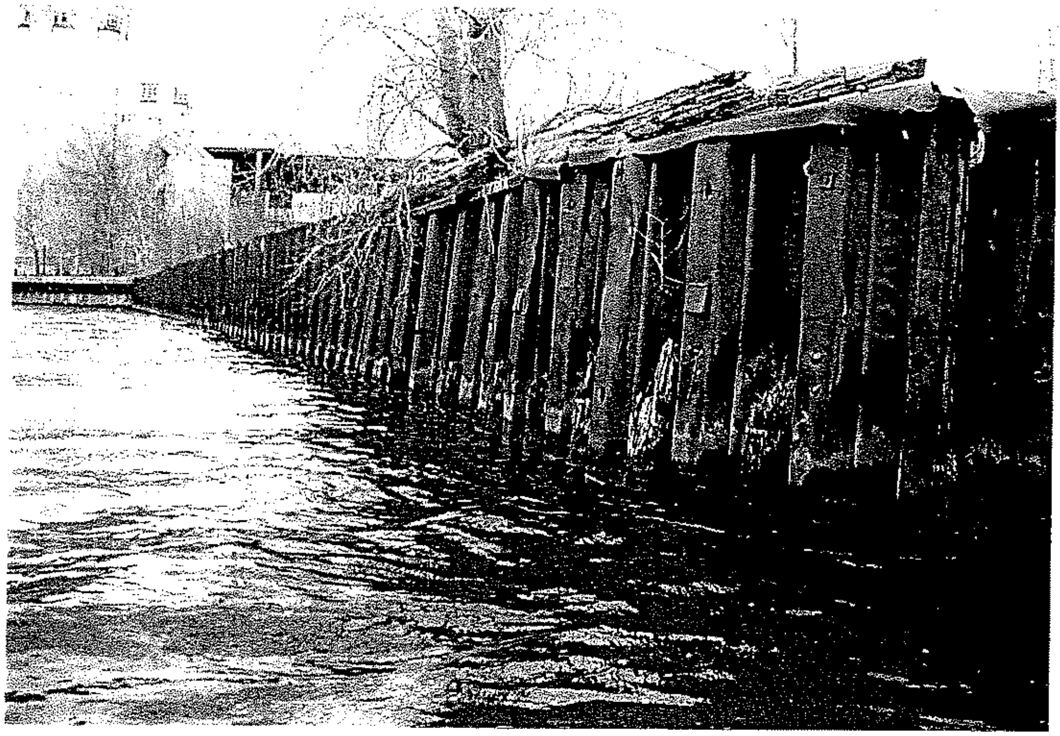
REPLACEMENT DOCKWALL SECTION

Drawn By: DR	COLLINS ENGINEERS INC. 123 North Wacker Drive, Suite 300, Chicago, IL 60606, (312) 704-9300, www.collinsengr.com	Date: AUGUST, 2005
Checked By: JEO	ILLINOIS PROFESSIONAL DESIGN FIRM LICENSE NO. 184-000993	Scale:
Code: 43720001		Figure No.: 7

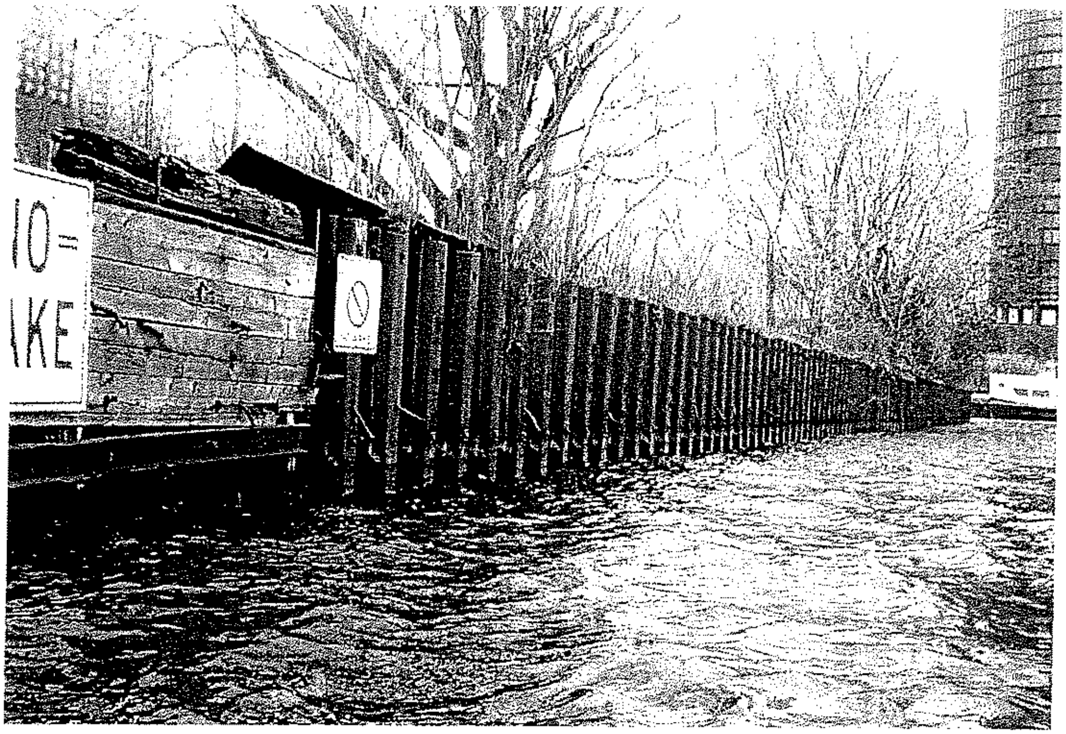
FILE: g:\4372\43720001.dgn

Appendix B

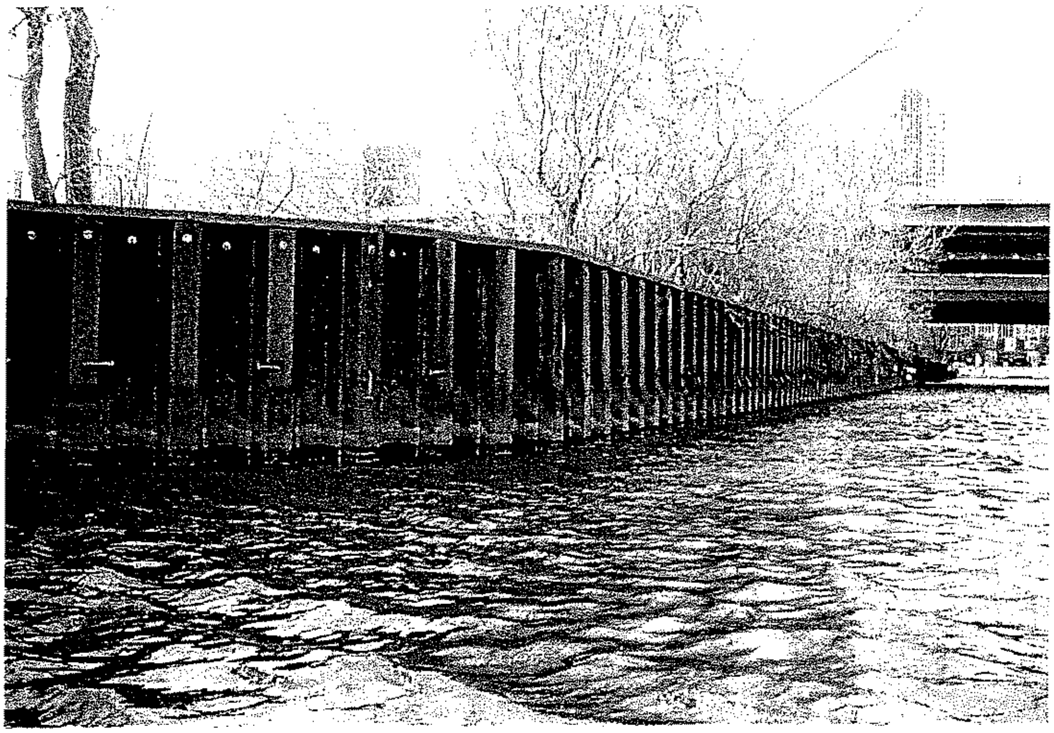
Photographs



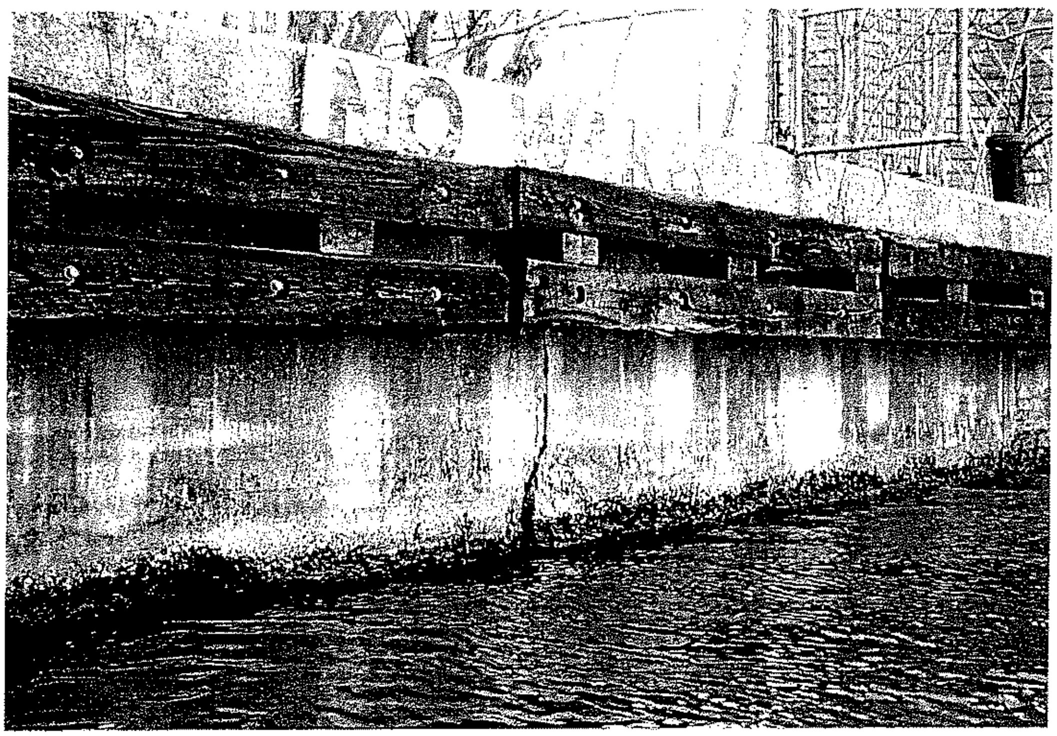
Photograph 1. Overall View of South Dockwall Face, Looking Northwest from Station 3+66.



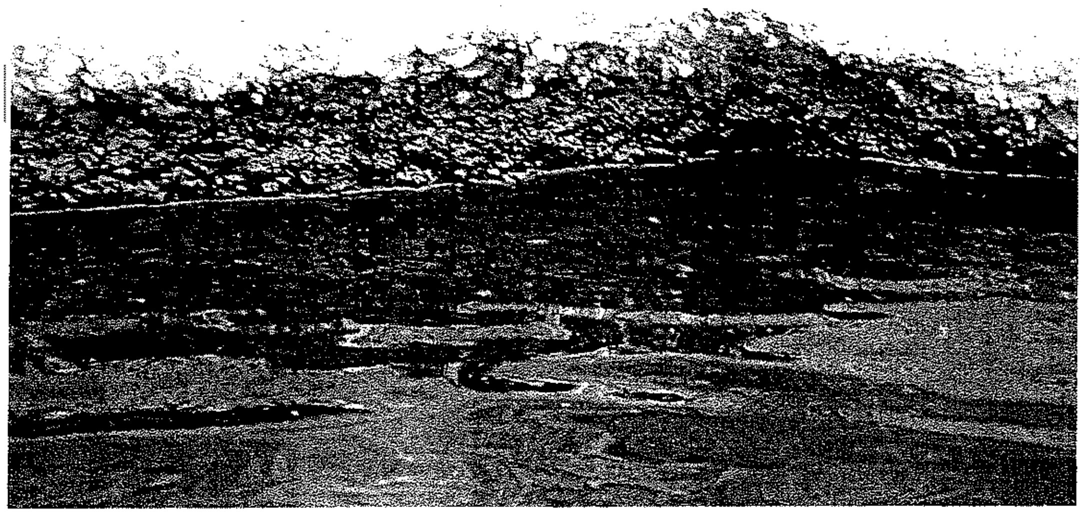
Photograph 2. Overall View of East Dockwall Face, Looking Northwest from Station 3+66.



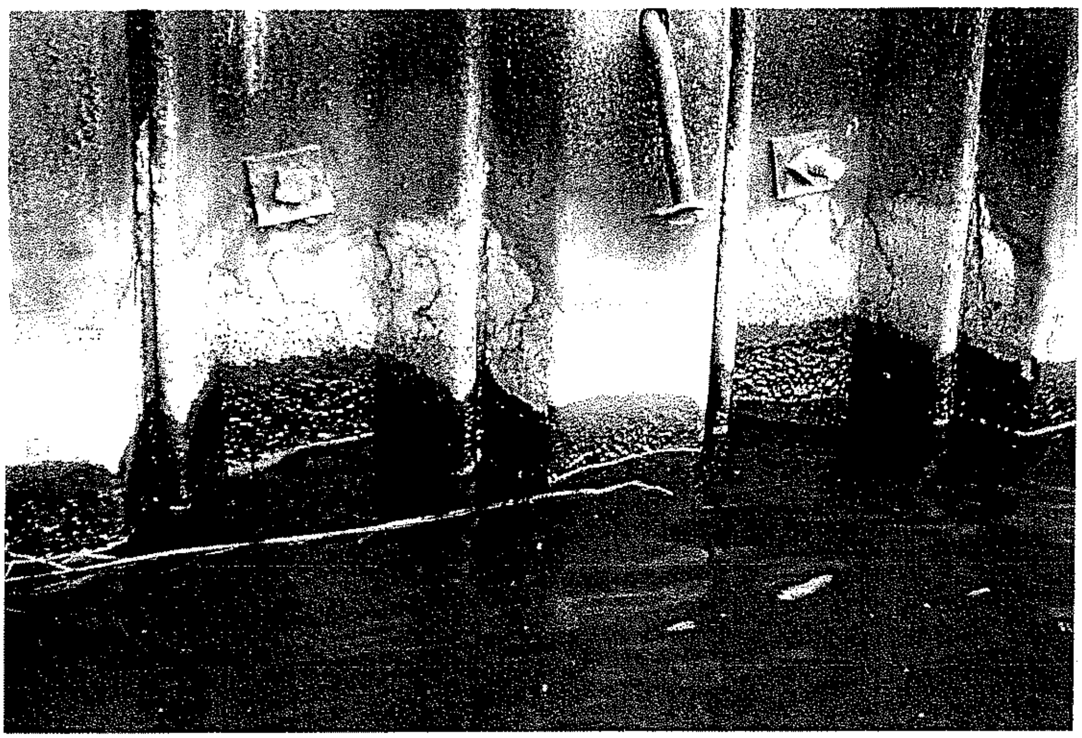
Photograph 3. Overall View of North Dockwall Face, Looking Southwest from Station 7+77.



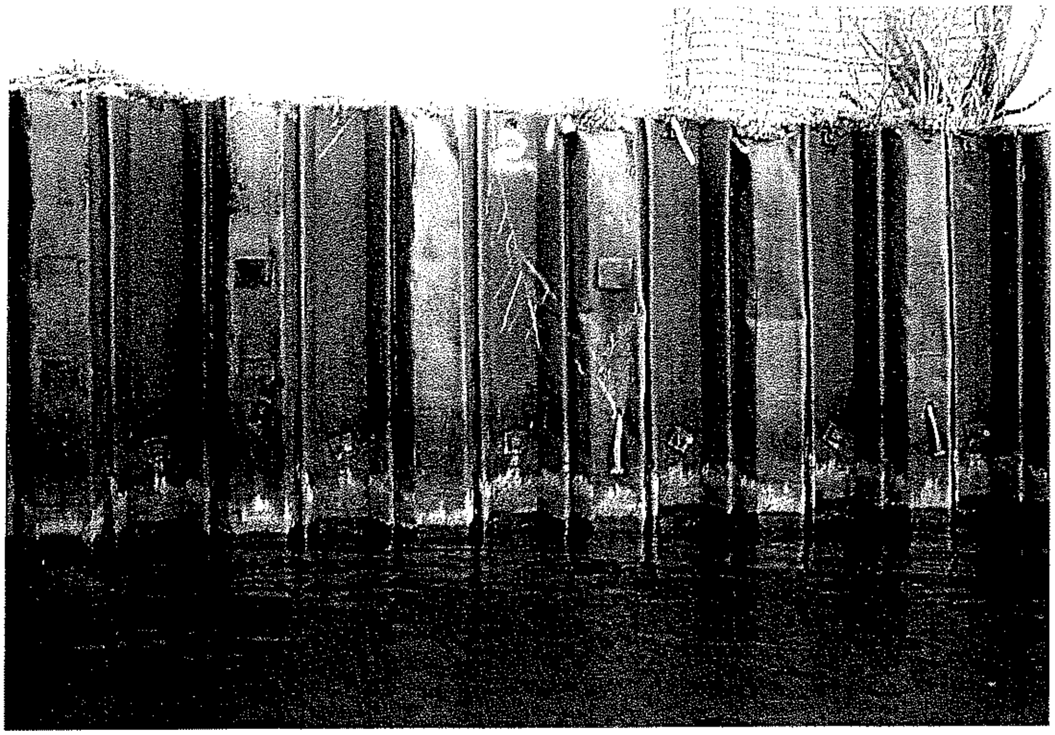
Photograph 4. Dockwall at Station 0+30, Looking Northeast.



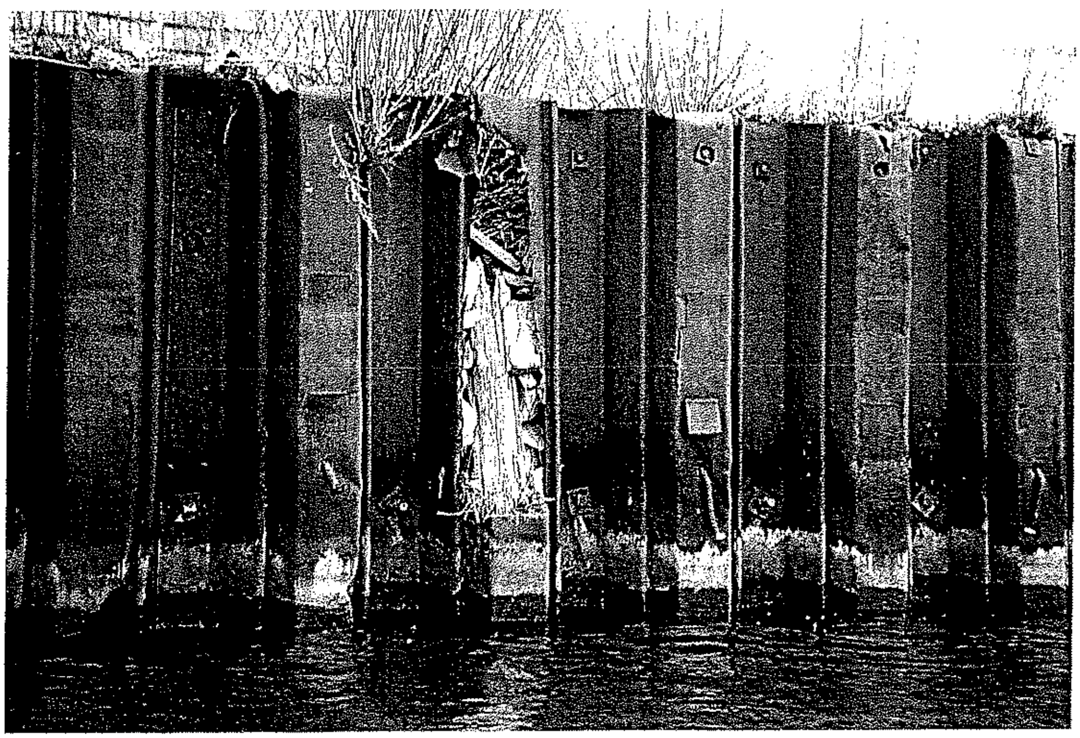
Photograph 5. View of Typical Concrete Cap Condition along Waterline at Station 0+20, Looking North.



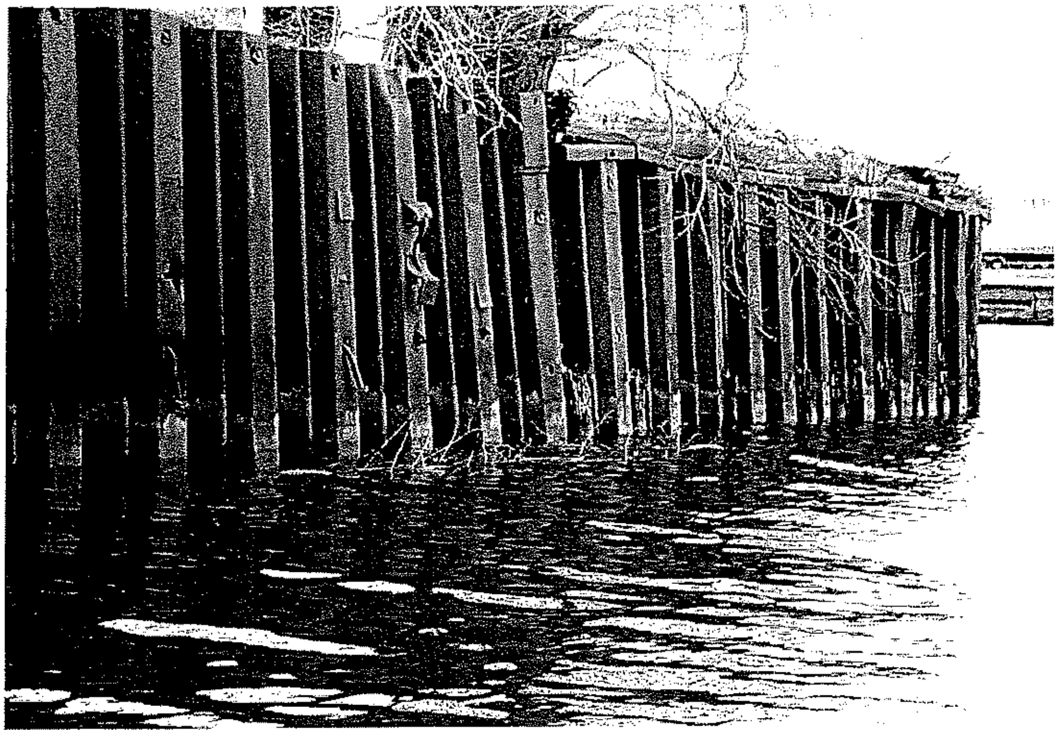
Photograph 6. View of Typical Steel Condition along Waterline at Station 0+75, Looking North.



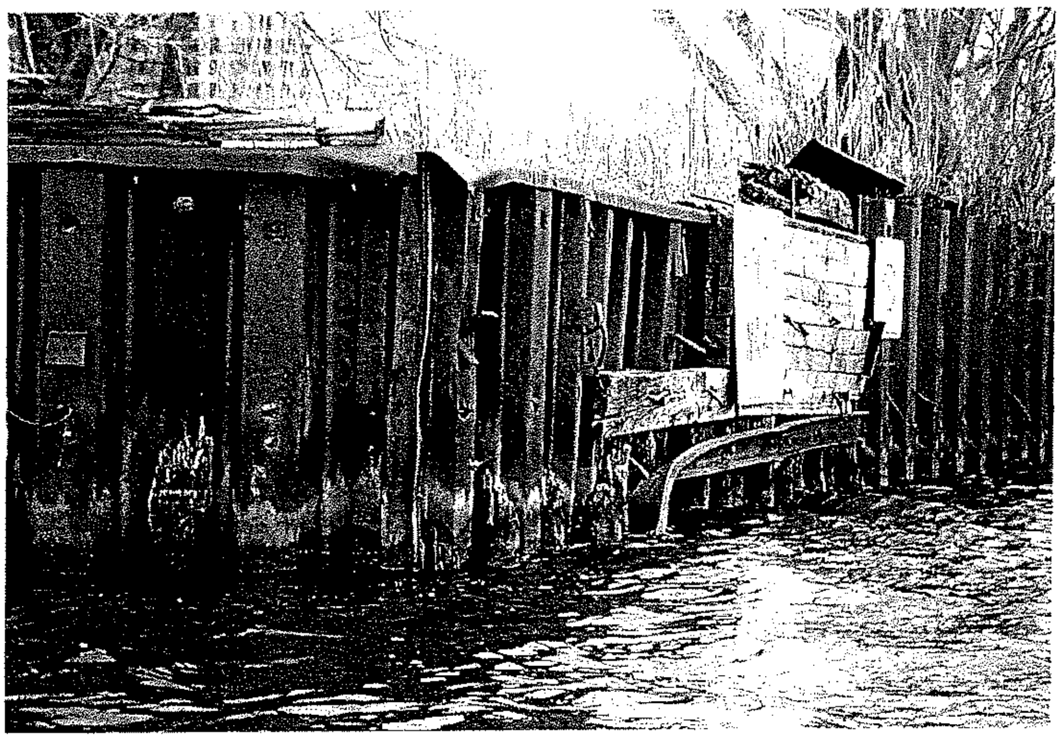
Photograph 7. View of Dockwall at Station 2+00, Looking North.



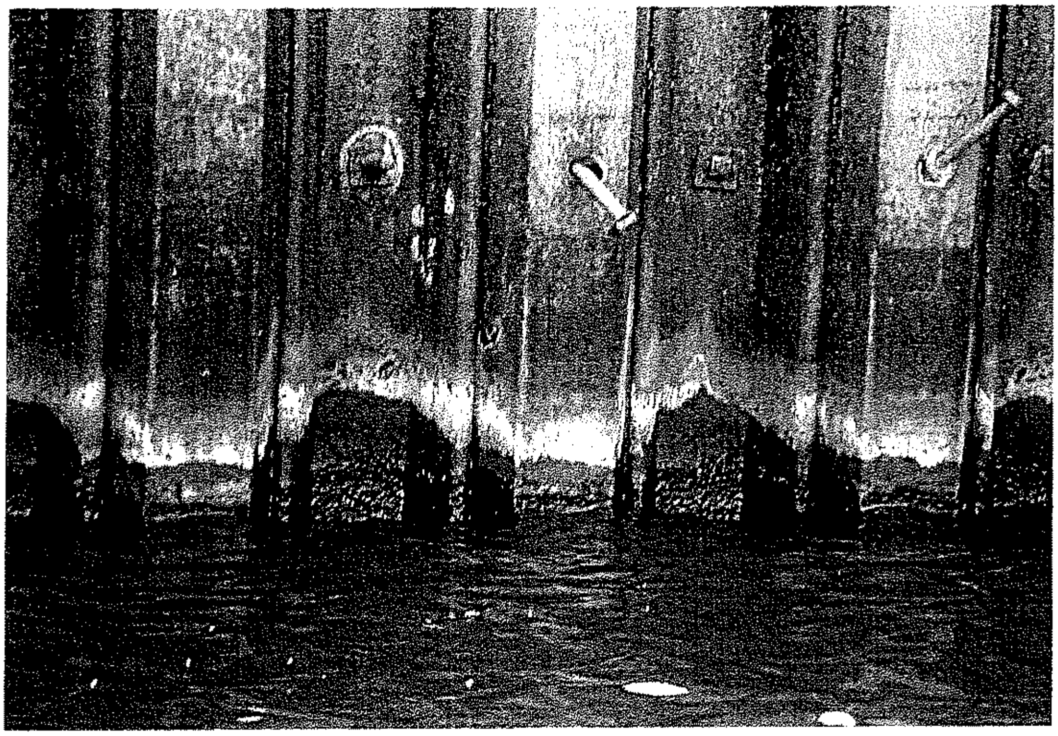
Photograph 8. View of Failed Sheet at Station 2+80, Looking North.



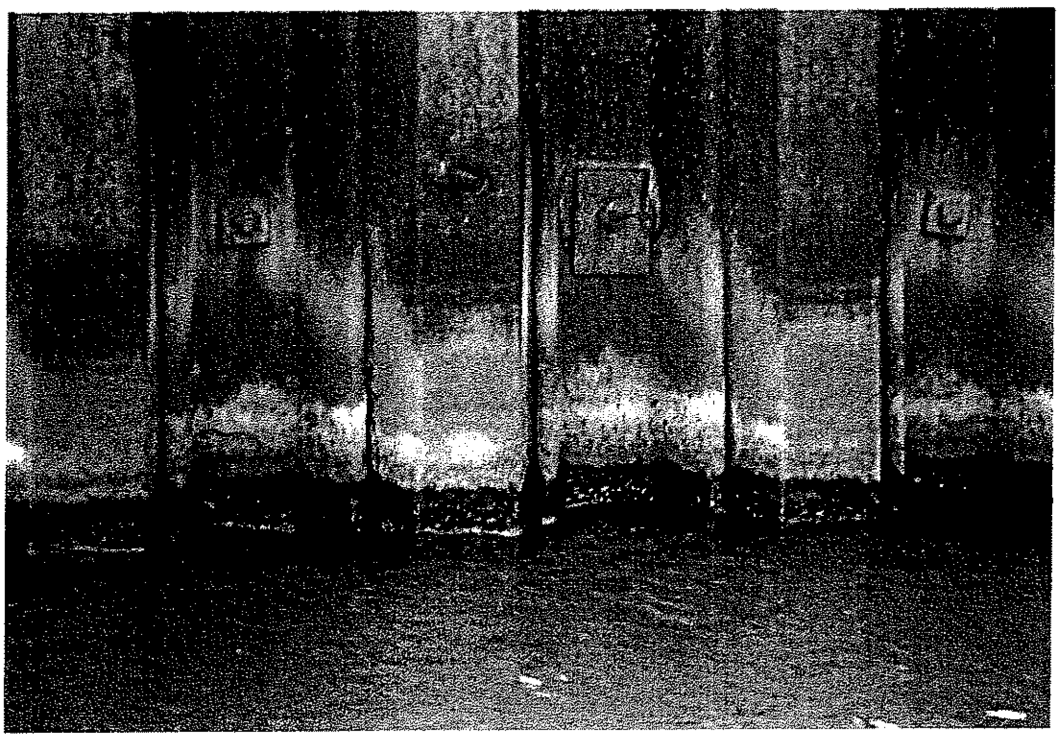
Photograph 9. View of Typical Dockwall Configuration, Looking Northeast from Station 3+00.



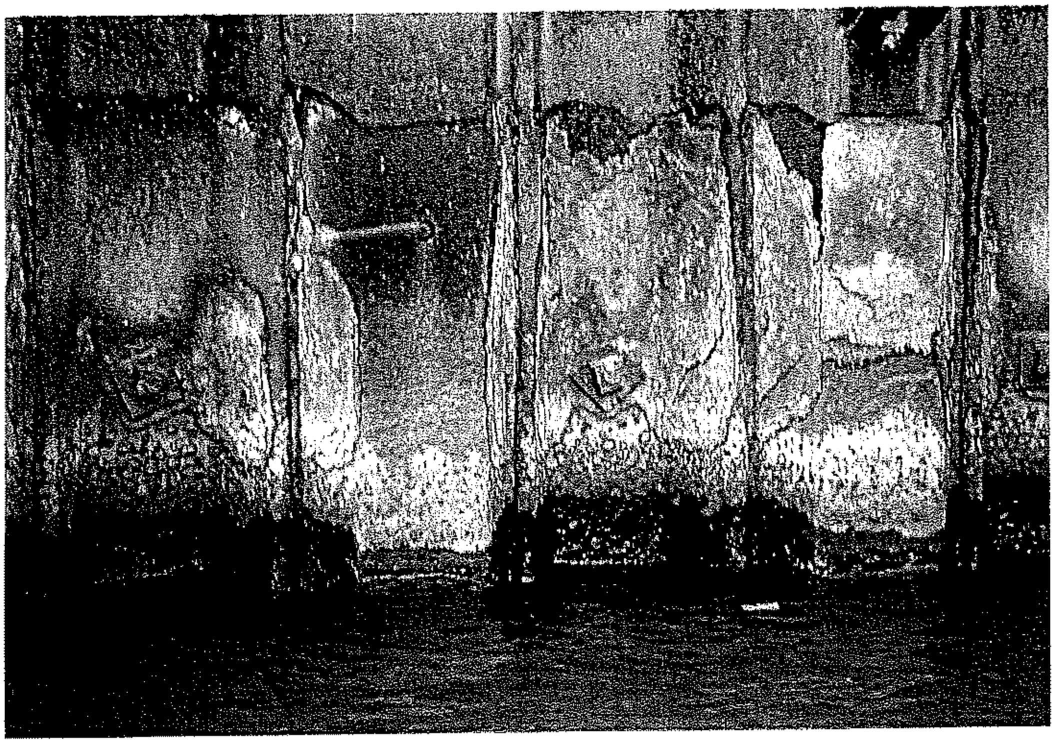
Photograph 10. View of Typical Steel Condition, Looking Northwest from Station 3+66.



Photograph 11. View of Typical Steel Condition at Station 7+00, Looking West.

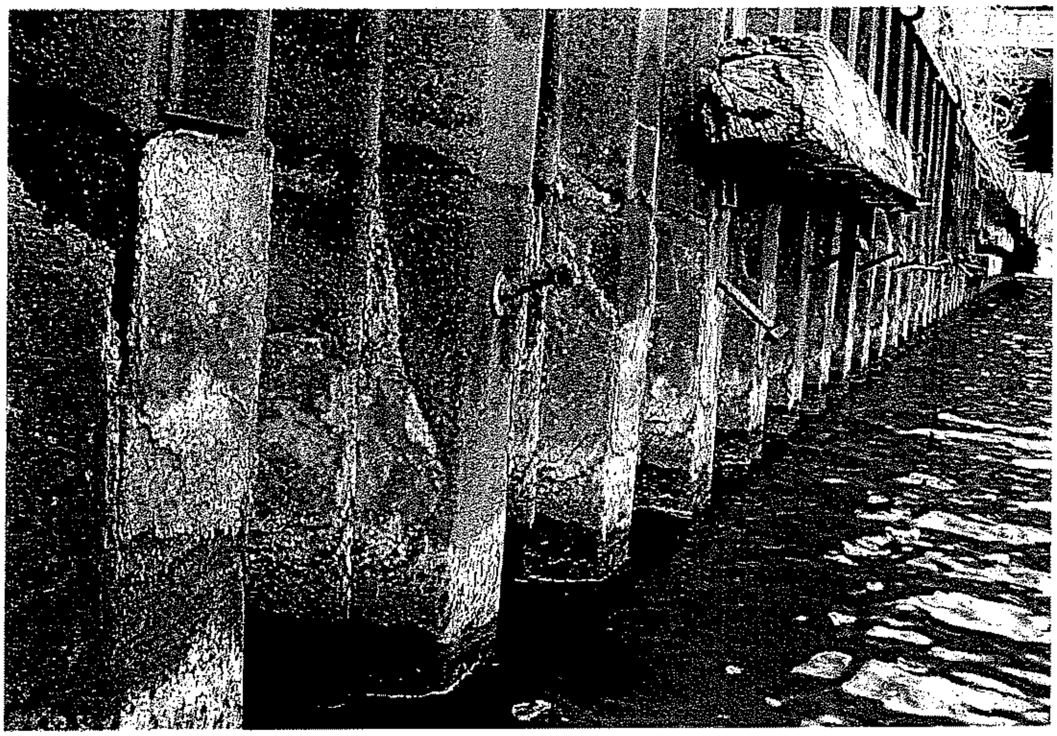


Photograph 12. View of Typical Steel Condition at Station 8+00, Looking West.



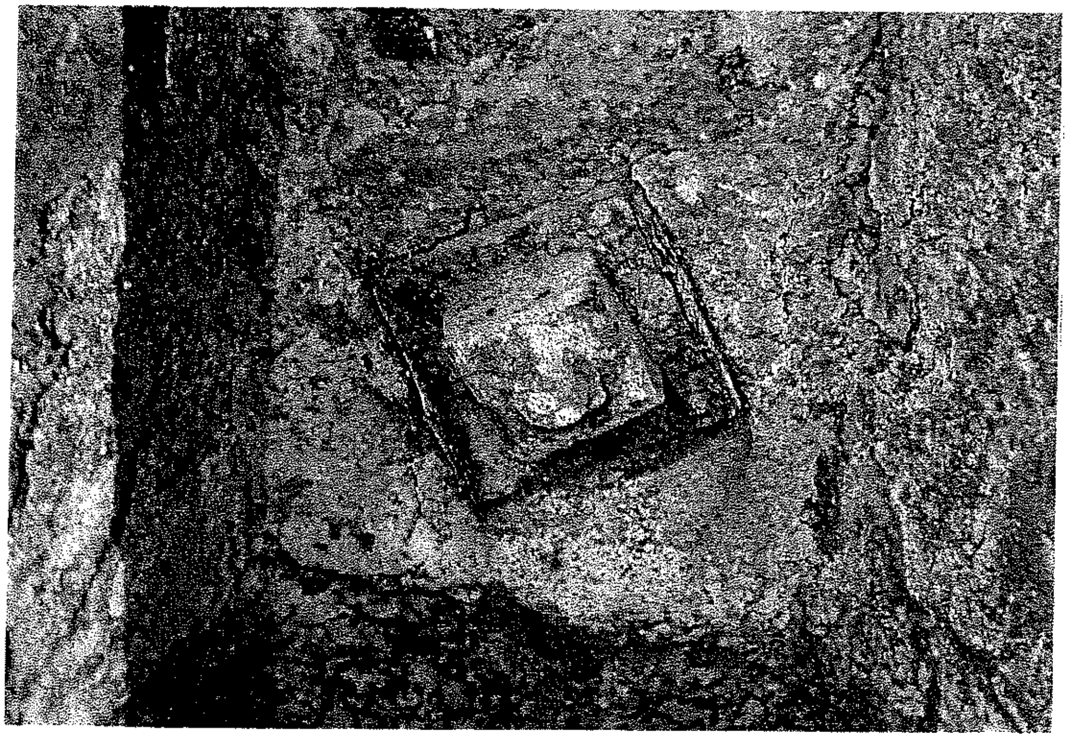
Photograph 13. View of Typical Steel Condition at Station 9+40, Looking South. Note Heavy Layer of Pack Rust and Steel Section Loss from the Waterline up 2 Feet.

Photograph 14. View of Typical Steel Condition at Station 9+40, Looking South. Note Heavy Layer of Pack Rust and Steel Section Loss from the Waterline up 2 Feet.

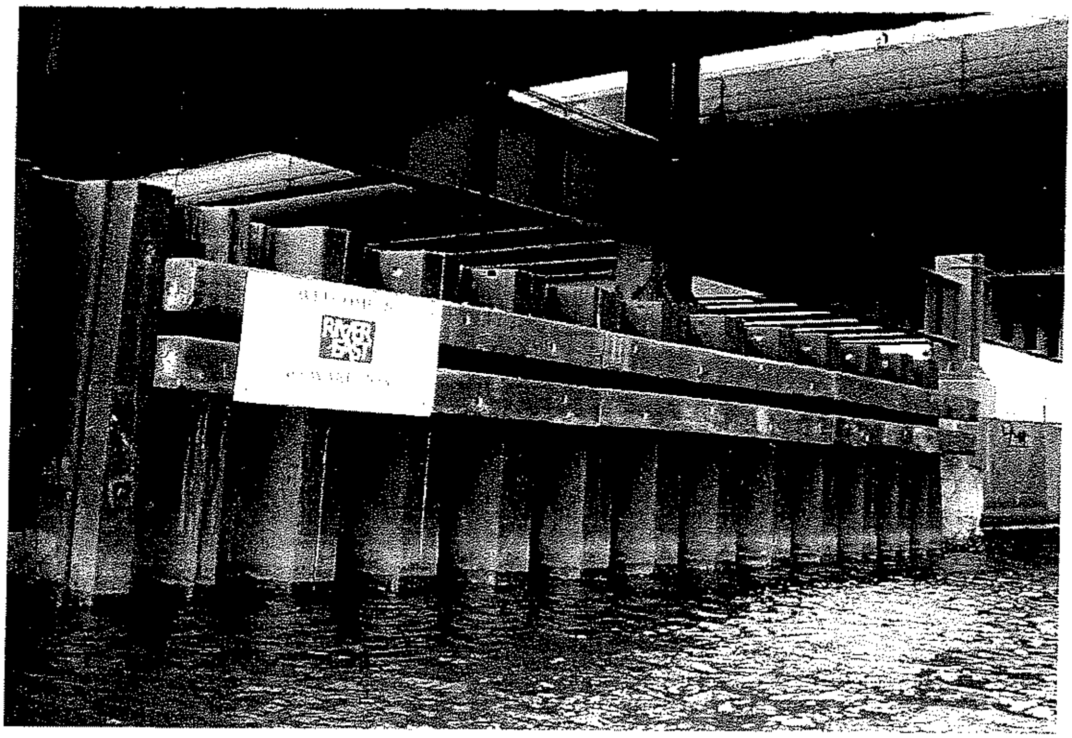


Photograph 15. View of Typical Steel Condition, Looking Southwest from Station 9+40.

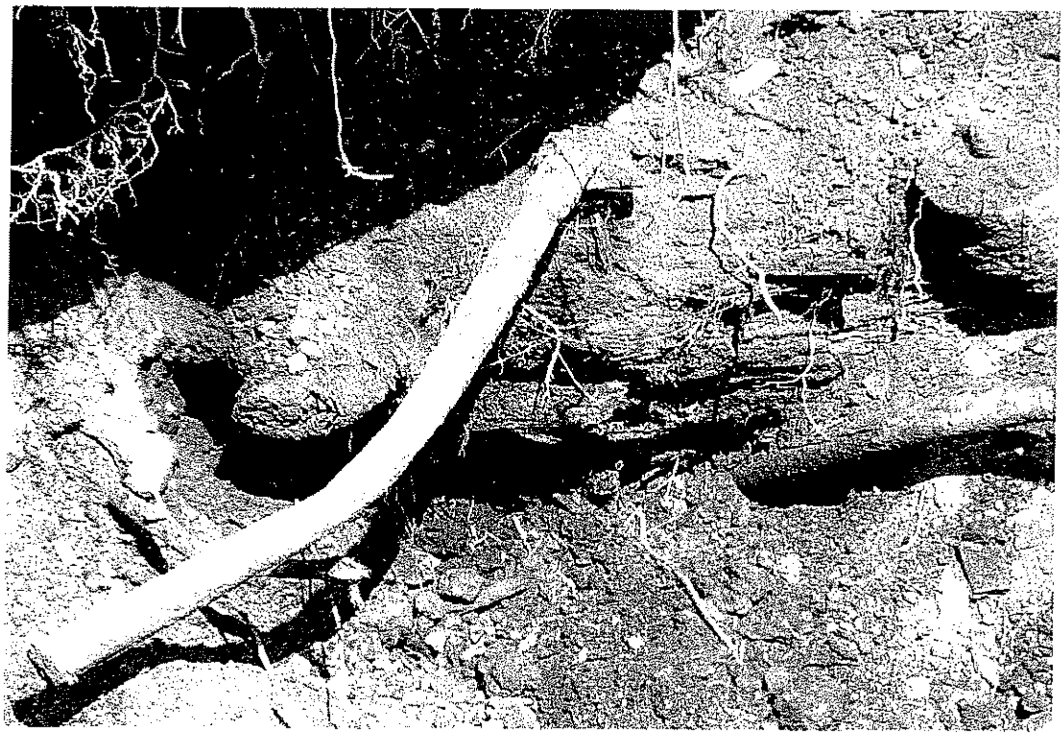
Photograph 16. View of Typical Steel Condition, Looking Southwest from Station 9+40. Note Heavy Steel Section Loss 2 Feet Above the Waterline.



Photograph 17. View of Typical anchor Condition at Station 9+80, Looking South.

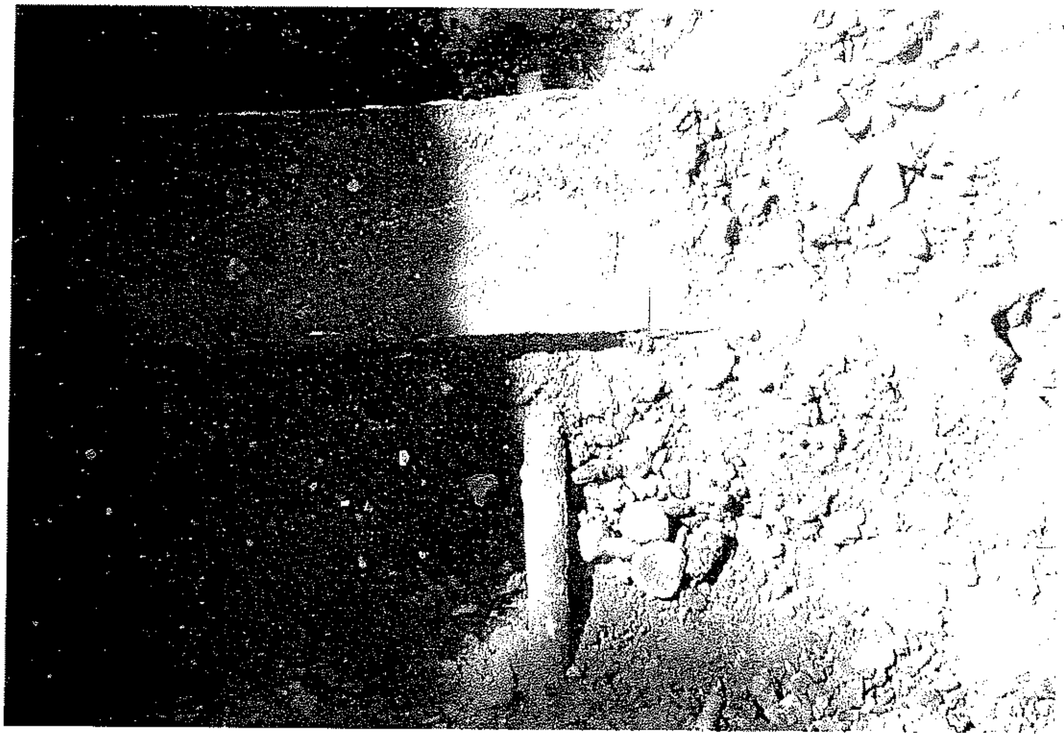


Photograph 18. View of Typical Dockwall Configuration, Looking Southwest from Station 10+85.



Photograph 21. View of Western Anchor Rod End at Station 3+94. Note Lack of Anchor Restraint System.

Photograph 22. View of Interior Steel Sheeting Face at Station 9+50.



Photograph 23. View of Anchor Rod to Channel Connection at Station 9+50.